EVALUATION AND ENHANCEMENT OF OPPOSED LEFT-TURN FLOW ANALYSIS METHODOLOGY

By

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Abstract of Dissertation Presented to the Graduate School of the University of Florida in Partial Fulfillment of the Requirements for the Degree of Doctor of Philosophy

EVALUATION AND ENHANCEMENT OF OPPOSED LEFT-TURN FLOW ANALYSIS METHODOLOGY

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Left-turn movements are a major source of traffic conflicts at signalized intersections. The existence of such movements aggravates traffic delays and safety problems. The saturation flow rate of permitted left-turn movements is an important factor in the operation of these intersections. Several analytical models have been developed to estimate the saturation flow rate for these movements. However, the sensitivity of these models to various timing and traffic demand conditions has not yet been adequately assessed. This is due to the fact that field data cannot pratically be accurately collected for all requisite variables at such intersections.

Using the TRAF-NETSIM simulation model, this study analyzed left-turn operations at signalized intersections with a pre-timed signal. From the simulation output reports, a special method to "collect" detailed traffic information was developed. In addition, analytical models were constructed to explicitly show the relationships among important variables.

These analytical models not only give a clear description of left-turn operations but also facilitate sensitivity studies of changes in traffic conditions and signal-timing schemes.

Since left-turn performance is believed to be germane to the left-turn saturation flow rate, methods for estimating the left-turn saturation flow rate were throughly reviewed. Because of the recognized inadequacies in existing methods, a new method that can yield a more reasonable estimate of the left-turn saturation flow rate was developed. The effects of cycle length, cycle split, the number of opposing lanes, and the headway distribution were addressed.

CHAPTER ONE

INTRODUCTION

Need for the Research

There are three types of left-turn phasing that are in general use at signalized intersections:

1. Permitted left turn. Vehicles are allowed to make a turn on a circular green indication, but must yield to opposing traffic.
2. Protected left turn. Vehicles are allowed to make a turn only on a green arrow indication and have the right-of-way while the green arrow is displayed.
3. Compound left-turn protection. Vehicles are allowed to make a turn either on a green arrow indication or on the circular green that they must yield to opposing traffic. This phasing includes the green arrow followed by the circular green (protected-plus-permitted) and the circular green followed by the green arrow (permitted-plus-protected), in either sequence.

Permitted left-turn phasing allows left turns to be made through gaps in the opposing traffic flow. The existence of such phasing can increase left-turn capacity and reduce overall intersection delay. Since permitted left turners have to wait for acceptable gaps or amber time to clear the intersection, it can be recognized that as the left-turn demand approaches a saturated condition, more left turners will queue up and incur excessive delay. If an exclusive left-turn lane is not provided, the left-turning queue also will impede through movements. When drivers suffer long delays, they may become impatient and make hazardous maneuvers. Although numerous accidents and complaints may indicate the need for some form of left-turn treatment such as an exclusive left-turn lane, a protected left-turn phase, or both, traffic engineers need clear guidelines to justify implementing a left-turn treatment. The problem is how to identify the critical condition of left-turn operations so that a treatment can be implemented.

Some researchers have tried to relate the critical condition to the left-turn saturation flow rate, which is the maximum number of left turns that can be made in one hour. The saturation flow rate of permitted left-turn lanes depends primarily on the opposing traffic, driver behavior, vehicle characteristics, the intersection geometric configuration, and the signal-timing scheme. Procedures for determining the left-turn saturation flow rate have been debated heatedly for decades. Until this controversy surrounding the left-turn saturation flow rate is settled, the left-turn critical condition cannot be determined effectively.

Among the existing methods, Chapter 9 of the Highway Capacity Manual (HCM) (Transportation Research Board, 1985) contains procedures for evaluating saturation flow rate, delay, and level-of-service of signalized intersections. Although the HCM disclaimer clearly states that it is not intended as a standard, it has become a de facto standard, and several state and local agencies have mandated its use.

When an analysis involves permitted left-turn lanes, the HCM uses a left-turn adjustment factor in the estimation of the prevailing saturation flow rate for each lane group. The determination of the left-turn adjustment factor relies on a complicated analytical model.

Questions concerning the reliability of this model have been raised (Hagen and Courage, 1989; Prassas and Roess, 1992; Bonneson, Messer, and Fambro, 1988).

The United States Department of Transportation, Federal Highway Administration (FHWA), has recognized the need for providing the traffic engineering community with a procedure for estimating the left-turn saturation flow rate. Several projects have been aimed at improving the procedure in the HCM. In early 1990 the results of an FHWA-sponsored project were reported (Roess, Ulerio, and Papayannoulis, 1990). The recommendations of the report included replacing the theoretical model for left-turn factors for shared, permitted-lane groups in the 1985 HCM with regression models. The credibility of these models is supported by the largest empirical database in existence today. These models were simpler than those in the 1985 HCM, but these models lacked the clear logic of the HCM theoretical approach. For example, for single-lane approaches, the conclusion that the left-turn saturation flow rate did not depend on the opposing flow rate was counterintuitive (Prassas and Roess, 1993).

In 1992 a hybrid method was developed by merging the regression models into the 1985 HCM framework and replacing the regression model for estimating the left-turn saturation flow rate with Messer and Fambros' (1977) left-turn equivalent model. This method was incorporated into the revised version of Chapter 9 of the HCM (1994 HCM) (Transportation Research Board, 1994). With the publication of the 1994 HCM, the new model has become the most widely used method in the U.S.A. for estimating the saturation flow rate of left-turn lanes. However, the degree of sensitivity of these models to various traffic conditions has not yet been adequately compared and evaluated. This is due to the fact that field data cannot practically be accurately collected for all requisite variables. Because of the absence of field data for model validation and the complexity of the traffic flow, simulation models offer a practical and economical alternative approach to left-turn flow analysis.

The Network Simulation (NETSIM) model was developed for the FHWA nearly 25 years ago and has been enhanced continuously since that time. NETSIM is a predominantly general-purpose intersection network simulator. The model also provides measurements of signalized intersection performance. There is a definite overlap in the analysis capabilities of NETSIM and those in HCM Chapter 9. Their simultaneous existence as alternative problem-solving methods raises legitimate questions in the minds of engineers as to their similarities and differences. Examples of such questions are:

1. how well does NETSIM match the HCM using default parameters?
2. how well does NETSIM compare with other models?
3. how well does HCM compare with other models?
4. how could HCM equations be changed to match NETSIM? and
5. how could NETSIM parameters be changed to match HCM?

Consequently, there is a need for further research to answer these questions.

Objectives

This research presents an enhanced methodology for estimating the saturation flow of permitted left-turns. The methodology accommodates both exclusive and shared left-turn lanes in addition to compound left-turn protection. The objectives of this study are to:

1. identify systematic differences between analytical and simulation models that would cause their results to differ;
2. investigate simulation model parameter adjustments that would improve the agreement with analytical models; and
3. develop an analytical method (i.e., combination of model elements) that produces a better match with simulation models.

The specific tasks of the research are to:

1. review the literature with respect to left-turn flow analysis;
2. identify the candidate data sets for comparison, augmented as necessary by hypothetical data;
3. identify and analyze the similarities and differences among the existing models, considering such factors as:
    a. definition of terms,
    b. model structures, and
    c. calibration of parameters,
4. identify shortcomings in the existing models;
5. investigate modeling improvements that will overcome these problems;
6. develop an enhanced methodology for estimating left-turn saturation flow rates; and
7. propose suggestions for specific changes to the HCM procedures.

Organization

This dissertation is organized according to the tasks stated previously. The next chapter is a review of the literature that is pertinent to this study, and it includes the methodologies for estimating the saturation flow rate of permitted left turns.

The study methodology for this dissertation is presented in Chapter Three. The intersection logic in NETSIM is evaluated first. A scenario that contains only thru movements is analyzed to explore how well the model simulates the traffic conditions without left-turn effects. Then the procedure for collecting simulation results is described. This procedure can allow researchers to obtain much more detailed traffic information than that contained in the standard NETSIM output report.

The enhancement of the left-turn model is presented in Chapter Four. The model elements for estimating the saturation flow rate of exclusive permitted left-turn lanes are addressed first. Then, the research extends to the model dealing with shared permitted left-turn lanes. Finally, all possible left-turn treatments are considered.

Chapter Five presents a methodology for estimating the saturation flow rate of permitted left-turn lanes. The proposed model is incorporated into HCM procedures for comparison with existing analytical and simulation models.

Chapter Six contains an intensive evaluation of the models. The proposed model, the 1985 HCM and the 1994 HCM methodologies are compared with simulation results.

Conclusions and recommendations emanating from this research effort are presented in the final chapter. The recommendations include suggestions for future research.

CHAPTER TWO

REVIEW OF OPPOSED LEFT-TURN FLOW ANALYSIS METHODOLOGY

The purpose of this section is to present a review of earlier research on the analysis of left-turn flow in order to avoid duplication and to stimulate new methodology development. This literature review covers three areas which are pertinent to this study. The first part focuses on the existing models for estimating the saturation flow rate of exclusive permitted left-turn lanes. The second part reviews the current methods for modeling shared left-turn lanes. The last part explores the algorithms for analyzing compound left-turn phase sequencing.

The following notations are used throughout the dissertation.

α = critical gap (sec);

c = capacity (vph);

C = cycle length (sec);

D = total delay (sec/veh);

E_{L1} = thru-vehicle equivalent for opposed left turns;

exp = natural base of logarithms, or $e^{(\text{exponent})}$;

F_i = the product of seven adjustment factors related respectively to lane width, heavy vehicles, approach grade, parking, blocking effects of local buses, area type, and right turns;

f_{LT} = adjustment factor for left turns;

f_m = left-turn adjustment factor applied to shared lane only;

g = effective green time (sec);

g_f = initial portion of green until arrival of first left-turning vehicles (sec);

g_o = green time for the opposing traffic (sec);

g_q = saturated green time for the opposing queue (sec);

g_u = unsaturated portion of green after opposing queue is cleared (sec);

h_l = mean minimum left-turn headway (sec);

h_o = mean minimum opposing headway (sec);

N = number of lanes in a lane group;

P_L = proportion of left-turn vehicles in the shared lanes.

q = opposing flow rate (vps);

s = saturation flow rate (vphg);

s_o = ideal saturation flow rate, taken to be 1,800 vph per lane;

s_u = left-turn saturation flow rate during unsaturated green time (vphg);

s_u' = left-turn saturation flow rate during unsaturated green time (vpsg);

v_o = adjusted opposing flow rate (vph);

vph = vehicles per hour;

vphg = vehicles per hour during green time;

vps = vehicles per second.

Exclusive Permitted Left-Turn Lanes

Extensive research has been conducted in the area of estimating the saturation flow rate of exclusive permitted left-turn lanes. A number of models are presented in the literature. These models can be classified as being one of three types: analytical, empirical, or simulation models. Each of these models is presented separately.

Existing Analytical Models

In the capacity analysis of a signalized intersection, the 1985 HCM requires that the saturation flow rate of a lane group be determined from the following formula:

$$s = s_o \times N \times F_i \times f_{LT} \tag{2.1}$$

For permitted turns made from an exclusive left-turn lane, the 1985 HCM computes f_{LT} based on the following model:

$$f_{LT} = \left(\frac{g_u}{g} \times \frac{1}{E_{L1}}\right) + \frac{4}{g} \tag{2.2}$$

The unsaturated portion of the green phase, g_u, is defined as

$$g_u = \frac{g - CY}{1 - Y} \tag{2.3}$$

where $Y = v_o/s_o$.

The through-vehicle equivalent for opposed left turns is

$$E_{L1} = \frac{1800}{1400 - v_o} \tag{2.4}$$

It can be shown that by substituting Equations (2.2) and (2.4) into Equation (2.1) and by assuming that N and F_i are all equal to 1, the following formula can be derived:

$$s = (1400-v_o)\frac{g_u}{g} + 2(\frac{3600}{g}) \tag{2.5}$$

The Australian Road Research Board (ARRB) model (Akçelik, 1989) uses a thru-car equivalent, e_o, to calculate the effective opposed turn saturation flow from

$$s = \frac{1800}{e_o} \tag{2.6}$$

where 1800 is the base saturation flow and e_o is given by

$$e_o = \frac{0.5g}{s_u^{/}g_u + n_s} \tag{2.7}$$

The two parameters, g and g_u, are the same as those defined in the 1985 HCM model. The opposed left-turn saturation flow (vps) is defined as

$$s_u^{/} = \frac{q \times e^{-5q}}{1-e^{-3q}} \tag{2.8}$$

Similarly, substituting Equation (2.7) into Equation (2.6) will give

$$s = (3600 \times s_u^{/} \times \frac{g_u}{g}) + (n_f \times \frac{3600}{g}) \tag{2.9}$$

Existing Empirical Models

In the Signal Operations Analysis Package (SOAP) (Hagen and Courage, 1989), the saturation flow rate for permitted left-turns is determined by the following models, which were based on data derived from simulation using the NETSIM model.

For a single opposing lane,

$$s_u = 1404 - 1.632 \times v_o + .0008347 \times v_o^2 - .0000002138 \times v_o^3 \tag{2.10}$$

For multiple opposing lanes,

$$s_u = 1393 - 1.734 \times v_o + .0009173 \times v_o^2 - .0000001955 \times v_o^3 \tag{2.11}$$

The unsaturated green time (g_u) is defined by the following equation in the SOAP:

$$g_u = g_o - g_q \tag{2.12}$$

In Equation (2.12), g_q is defined by

$$g_q = \frac{v_o(C-g_o)}{(s_o-v_o)} \tag{2.13}$$

This equation is exactly the same as the unsaturated green model for the HCM, provided that $g_o = g$.

Early in the year 1990 the results of a FHWA-sponsored project were reported in a Transportation Research Board (TRB) paper. The new models were developed from regression analysis of field data (Roess, Ulerio, and Papayannoulis, 1990). The regression models were simpler than those of the 1985 HCM but lacked the clear logic of the theoretical approach. For example, for single-lane approaches, the conclusion that the left-turn adjustment factor did not depend on the opposing flow rate was counterintuitive (Prassas and Roess, 1992).

In 1992 a hybrid method of merging the regression models into the HCM framework was approved for the 1994 update of Chapter 9 of the HCM. This method also developed the adjustment factor for left turns through an opposing queue on single-lane approaches.

The 1994 HCM model uses a hybrid model which is obtained by merging a regression model for the unsaturated green time (g_u) into the 1985 HCM model. The regression model is given as follows:

for single-lane approaches

$$g_u = g - (4.943\, v_{olc}^{0.762} qr_o^{1.061} - t_L) \tag{2.14}$$

for multilane approaches

$$g_u = g_o - \left(\frac{v_{olc} qr_o}{0.5 - [v_{olc}(1 - qr_o)/g_o]} - t_L\right) \tag{2.15}$$

where qr_o = opposing queue ratio, that is, the proportion of opposing flow rate originating in opposing queues, computed as $1 - R_{po}(g_o/C)$;

R_{po} = platoon ratio for the opposing flow, obtained from $P_o(C/g_o)$; and

P_o = proportion of all vehicles in the opposing flow arriving during the green phase.

The model for determining the unsaturated green time for multilane approach, Equation (2.15), can be simplified as below,

$$g_u = \frac{g_o - CY}{1 - R_{PO} Y} + t_L \tag{2.16}$$

This equation is exactly the same as that of the 1985 HCM, Equation (2.3), if R_{po} is equal to 1 and t_L is equal to zero.

Like the 1985 HCM model, the 1994 HCM model uses Equation (2.1) to calculate the saturation flow rate. However, the f_{LT} is defined as

$$f_{LT} = \frac{g_u}{g} \times \frac{1}{E_{L1}} \tag{2.17}$$

where E_{L1} is a set of values developed by Messer and Fambro (1977). Table 2.1 lists the values for exclusive permitted left-turn lanes.

Table 2.1. Thru-car equivalents, E_{L1}, for permitted left turns.

No. of signal phases	Type of left-turn lane	No. of opposing lanes	E_{L1} by opposing flow, v_o						
			0	200	400	600	800	1000	≥1200
2-phase	Exclusive	1	1.05	1.7	2.6	4.7	10.4*	10.4*	10.4*
		2	1.05	1.6	2.2	2.9	4.1	6.2	10.4*
		≥3	1.05	1.6	2.1	2.8	3.6	4.8	10.4*
More than 2-phase	Exclusive	1	1.05	1.8	3.3	8.2*	8.2*	8.2*	8.2*
		2	1.05	1.7	2.4	3.6	5.9	8.2*	8.2*
		≥3	1.05	1.7	2.4	3.3	4.6	6.8	8.2*

* Generally indicates turning capacity only at end of phase-"sneakers" only.
(Source: HCM 1994, Figure 9.7.)

Thus, substituting Equation (2.17) into Equation (2.1) gives

$$s = \frac{1800}{E_{L1}} \times \frac{g_u}{g} \tag{2.18}$$

Comparison of Analytical and Empirical Models

It may appear at first sight that the existing models are based on different modeling structures; however, their basic two-term equations become obvious when they are substituted into the following general capacity equation:

$$c = s\frac{g}{C} \tag{2.19}$$

Specifically, for the 1985 HCM model,

$$c = (1400 - v_o)\frac{g_u}{C} + 2(\frac{3600}{C}) \tag{2.20}$$

and for the ARRB model,

$$c = 3600\, s_u / \frac{g_u}{C} + n_f(\frac{3600}{C}) \tag{2.21}$$

Lastly, for the 1994 HCM model,

$$c = \frac{1800\, g_u}{E_{L1}\, C} \tag{2.22}$$

The above derivation shows that all three models are based on the following two-term equation:

$$c = (s_u \times \frac{g_u}{C}) + \text{sneakers} \tag{2.23}$$

Both the 1985 HCM and the ARRB models use the first term to represent the opposed-turn capacity during the unsaturated green time (g_u) and the second term to indicate the number of sneakers. The 1985 HCM model assumes two sneakers per cycle. The ARRB model allows users to specify the number of sneakers (n_s). The 1994 HCM model considers only the opposed left-turn capacity during the unsaturated green time and assumes there are no sneakers. Table 2.2 shows a term-by-term comparison of these models.

From the above analysis, it is determined that the opposed left-turn capacity is the major concern for this study. Specifically, the opposed left-turn saturation flow, s_u, and the unsaturated green time, g_u, are the critical parameters for estimating the saturation flow of exclusive permitted left turns.

Table 2.2. Term-by-term model comparison.

Terms		1985 HCM	ARRB	1994 HCM
1	Opposed turn saturation flow, s_u	$1400 - v_o$	$\frac{3600 \times qe^{-5q}}{1 - e^{-3q}}$	$\frac{1800}{E_{L1}}$
	Unsaturated green time, g_u	$\frac{g - CY}{1 - Y}$	$\frac{g - CY}{1 - Y}$	$\frac{g - CY}{1 - R_{PO} Y}$
				$g - (4.943 v_{olc}^{0.762} qr_o^{1.061} - t_L)$
2	Number of sneakers	2	n_s	0

There are many models for predicting s_u. The most widely known were considered in this study and are summarized in Table 2.3.

Tanner's (1962) model was initially derived for a single lane of opposing vehicles, but it was proposed that the condition of multilane opposing vehicles could be approximated by regarding the opposing vehicles as a single stream with an arrival rate twice that of a single lane and one-half the minimum headway. Webster and Cobbe (1966) used these theoretical equations in developing curves for estimating left-turn saturation flow rates. Estimations of the model parameters such as critical gaps and minimum headways were based on data collected in the field and on the test track. However, validation of these curves with field data is still lacking.

Drew (1968) derived an equation based on the analysis of gap-acceptance behavior for drivers merging at freeway ramps. Since this model resembles the condition under question, it was later proposed for estimating the left-turn capacity. Using observations to obtain appropriate values for two of these independent variables in Drew's equation, Fambro, Messed, and Anderson (1977) suggested a simplified form of the model, which is also presented in Table 2.3.

Simulation Models

Three simulation models are commonly used. They are TRANSYT-7F, the TEXAS model, and NETSIM.

TRANSYT-7F (Wallace, Courage, and Hadi, 1991) is a macroscopic, deterministic simulation and optimization model. In modeling the permitted movement, TRANSYT-7F accumulates arrivals on a step-by-step basis and releases the left-turn vehicles after considering the instantaneous volume of the opposing movement. The permitted-movement model (FHWA/TRC model) from releases 4-6 of the program was based on the field calibration and subsequent regression analysis. The following general exponential model was used:

$$MFR(t)_i = A_i \times \exp[-B_i \times v_o(t)^{C_i}] \tag{2.24}$$

where $MFR(t)_I$ = the maximum flow rate for permitted traffic at time t for condition I,

A_i = statistically derived intercept MFR at zero opposing flow for condition I,

B_i, C_i = statistically derived model parameters, and

$v_o(t)$ = the opposing flow rate at time t.

Table 2.3. Left-turn saturation flow rate formulations.

Model	No. of opposing lanes	Equation
Tanner	One	$s_u = [v_o(1-h_oq)]/\{\exp[q(\alpha-h_o)][1-\exp(-h_1q)]\}$
Tanner	Two or more	$s_u = [2v_o(1-h_oq)]/\{\exp[2q(\alpha-h_o)][1-\exp(-2h_1q)]\}$
Webster	One	$s_u = [v_o(1-(3)q)]/\{\exp[q(5-3)][1-\exp(-2.5q)]\}$
Webster	Two or more	$s_u = [2v_o(1-(1)q)]/\exp[2q(6-1/2(1))]\{1-\exp[-2(2.5)q]\}$
Drew	Any	$s_u = v_o\{[\exp(-q\alpha)]/[1-\exp(-qh_1)]\}$
Fambro, Messer, Anderson	Any	$s_u = v_o\{[\exp-q(4.5)]/[1-\exp(-2.5q)]\}$
1985 HCM	Any	$s_u = 1400 - v_o$
Australian	Any	$s_u = v_o\{[\exp-q(5)]/[1-\exp(-3q)]\}$
FHWA/TRC	One	$s_u = 1227[\exp(-.0244v_o^{.71})]$
FHWA/TRC	Two or more	for opposing speed < 40 mph $s_u = 1436[\exp(-.00066v_o^{1.22})]$ for opposing speed ≥ 40 mph $s_u = 1649[\exp(-.0101v_o^{0.83})]$
Ohio State	One	$s_u = 1404-1.632v_o+.0008347v_o^2-2.138\times10^{-7}v_o^3$
Ohio State	Two or more	$s_u = 1393-1.734v_o+.0009173v_o^2-1.955\times10^{-7}v_o^3$
Minnesota Model	One	$s_u = 1165-1.245v_o +1.4\times10^{-5}v_o^2\alpha$
Minnesota Model	Two or more	$s_u = 1145-.875v_o +1.2\times10^{-5}v_o^2\alpha$

Different sets of model coefficients were developed with respect to different phasing conditions, number of opposing lanes, and speeds. Even though this model was based on field data, it is consistently more pessimistic than other models commonly used. Release 7 uses the ARRB model as the default; in addition, users can overwrite this default model and use any of the six models provided by the program. These six models are the FHWA/TRC model, the ARRB model, the Texas A&M model, the Minnesota model, the Ohio State model, and the 1985 HCM model, all presented in Table 2.3.

The TEXAS model is a microscopic traffic simulation package consisting of a geometry processor, a driver-vehicle processor, and a traffic-simulation processor. The geometry processor calculates the paths that vehicles will follow, intersection conflicts, and available sight distance. The driver-vehicle processor generates random descriptors of the driver-vehicle units that are used in the simulation. The traffic-simulation processor propagates each driver-vehicle unit through the intersection system while gathering a large selection of performance statistics as output. The TEXAS model allows assessment of the effects of changes in road geometry, driver and vehicle characteristics, flow conditions, intersection control, lane control, and signal-timing schemes upon traffic operations. The model provides important information regarding left-turn operations such as opposing headway distributions, gap-acceptance criteria, and left-turn delay distributions.

TRAF-NETSIM[1] was developed for the FHWA nearly 25 years ago and has been enhanced continuously since that time. NETSIM is a microscopic, stochastic simulation model. It is designed primarily to provide the engineer with a range of traffic control and surveillance concepts for complex street networks.

The NETSIM simulation program describes in detail the operational performance of vehicles traveling over a network of urban streets. The model is based on a fixed-time, discrete-event simulation approach describing the dynamics of the traffic operations in urban street networks. The vehicles are represented individually and their operational performance is determined uniquely every second. Furthermore, each vehicle is identified by a category (auto, carpool, bus, truck) and by a driver behavioral characteristic (passive, normal, aggressive).

The program models turning vehicles in considerable detail. When the traffic signal turns from red to green, a left-turning vehicle is allowed to "jump" across the intersection before the opposing traffic starts. This is done by assigning a probability (user-specified or default) to the vehicle, representing its possibility of jumping. During the green signal indication, a left-turn vehicle in conflict with opposing traffic will wait for an acceptable gap (user-specified or defaulted) before making the turn. When the signal turns yellow, a left-turn vehicle that is waiting at the stopline is allowed to execute the turn. An execution speed of

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[1] The initial version of NETSIM was released in 1971 and was subsequently updated in 1973 and 1978. The model later became a component of the integrated traffic simulation system, TRAF, in the early 1980s, and was referred to as TRAF-NETSIM. However, now that the TRAF family is more fully populated, "NETSIM" is sufficient and is used in this dissertation.

22 feet/second is used for all left turns. If the turn is in conflict with pedestrians, delay will be incurred.

Shared Left-Turn Lanes

The model of the left-turn adjustment factor reflects the shared-lane operation in terms of three components. These components, as they occur from the commencement of green, are as follows:

Period 1	an initial portion, wherein some thru vehicles can proceed before the first, blocking left-turn vehicle arrives at the head of the queue,
Period 2	an interval subsequent to the clearance of the opposing queue wherein both thru and left-turn vehicles can depart, and
Period 3	a final period wherein left-turn vehicles clear the intersection before the initiation of the cross-street phase.

For permitted turns made from a shared left-turn lane, the 1985 HCM computes f_{LT} based on the following model:

$$f_{LT} = [f_m + (N-1)]/N \tag{2.25}$$

where

$$f_m = \frac{g_f}{g} + [(\frac{g_u}{g})\frac{1}{1+P_L(E_L-1)}] + \frac{2}{g}(1+P_L) \tag{2.26}$$

Benison et al. (1988) proposed a modification for the 1985 HCM procedure. This modification was directed toward an iterative approach wherein the values of s_u and P_L were recalculated based on the previous result. This procedure has the advantage of being able to incorporate better estimates of s_u and P_L during each iteration, which is particularly important in those instances where the opposing approach also has shared-lane operations.

The ARRB model is based on the same philosophy as the HCM. However, there are significant differences in the procedures. For period 1, the ARRB uses a generalized shared-lane model that predicts capacities before lane blockage using the model given by Hegarty and Pretty (1982). A "lane interaction" method also is used to model a shared-lane operation. In this method, a shared lane is considered to belong to two different movements that may block each other or depart together at different times (Akçelik, 1988). For period 2, the ARRB method uses a gap-acceptance formula to estimate S_u. This formula takes individual lane flows into account (Akçelik, 1989).

Additionally, TRANSYT-7F estimates the capacity of shared permitted left-turn lanes by using three models of saturation flow rate to simulate three different iterations. In the first iteration all links are simulated, but permitted links are modeled with a uniform maximum flow rate based on the HCM. In the second iteration only permitted links are simulated, using the permitted-movement model reported by Wallace (1987). Finally, in the third iteration all non permitted links are simulated to correct their flow patterns for any changes which occurred as a result of the second iteration. This procedure divides the available shared-lane capacity among its tenants on the basis of equivalent thru passenger vehicles (ETVs).

Based on the HCM structure of having three periods, Lin (1992) added another period at the beginning of the permitted phase to determine the capacity of the left turns made in front of the leading opposing vehicles (jumpers). Lin also introduced several probability equations to model these four periods.

The focus of previous research in this area has been on developing a model for estimating the value of period 1, g_f, the initial portion of green until the arrival of the first left-turning vehicle.

Compound Left-turn Protection

Compound left-turn protection is referred to as protected-plus-permitted or permitted-plus-protected left-turn phase in this dissertation. Studies in compound left-turn protection are limited. Most of these studies have used accidents and delay as criteria for selecting among three types of left-turn phasing: permitted, protected, and compound left-turn protection.

Agent and Deen (1979) surveyed 45 states to assess their procedures to determine left-turn treatment. Machemehl and Lin (1982) developed an elaborate set of measures of performance to aid in the selection of left-turn phasing. A series of monographs was developed, based on TEXAS Model simulations (Lee et al., 1977), as guidelines for whether or not protected left turns are justified.

Bonneson and McCoy (1987) re-analyzed Calculation 3 of the 1985 HCM. They have made an important contribution toward clarifying the confusing aspects of the methodology. These include the calculation of unsaturated green time and capacity of the permitted phase portion, assignment of left-turn volume to each phase portion, and the general iterative nature of the capacity analysis. Moreover, Bonneson and McCoy identified some areas that would appear to need revision. These include the calculation of lost time and delay with regard to compound left-turn protection movements.

Additionally, Hagen and Courage (1989) proposed a queuing model to account for all components of the left turns made on both a protected phase and a permitted phase. The queuing model offers two important advantages over the 1985 HCM methodology:

1. It eliminates the need for user-specified volume split.
2. It differentiates between leading and lagging protection.

Collins (1988) compared delay to left-turning vehicles under two different lead/lag phasing arrangements, specifically, the conventional compound left-turn protection and a special lead/lag permitted operation (also commonly known as "Dallas phasing"). Dallas phasing differs from conventional compound left-turn protection in that permitted left turns are allowed when the opposing left turn is protected, thus avoiding the yellow trap. Using the Texas simulation model for six different cases (which are unique combinations of cycle lengths and phase length arrangements), Collins tested several traffic volumes for each case. The following observations were made regarding the two different lead/lag arrangements:

1. The Dallas phasing produces less left-turn delay.
2. A significant reduction in the left-turn delay is observed for the Dallas phasing for opposing volumes less than 1000 vph (above this volume reduction in delay is small, as there are not enough gaps for left turners to filter through).
3. Reduced delay for vehicles in the direction with the leading protected turn of Dallas phasing is more significant than for those with lagging protection.
4. Benefits which can be realized from the leading and lagging phases of Dallas phasing are dependent on the magnitude of the opposing volume and the g/C for the thru traffic.

Fambro and Gaston (1991) also studied Dallas phasing and developed modeling parameters for use with this type of phasing. Field data were collected at four sites in Dallas during peak and off-peak periods. Four hours of observations were made during both peak and off-peak periods for each intersection using video, electronic, and manual data collection systems. The PASSER II program was used to evaluate the 360 different combinations of two left-turn phase sequences (Dallas and conventional lead/lag), two cycle lengths (90 and 120 seconds), three g/C ratios (0.4, 0.5, and 0.6), five left-turn volumes (100-300 vph in 50 vph increments), and six opposing volumes (300-800 vph per lane in 100 vph increments). Fambro's findings were generally in agreement with the conclusions in Collins' study.

The studies reviewed herein have largely concentrated on whether a protected left-turn phase is to be used, and, if so, the type of phasing to use (protected only or compound left-turn protection). The above studies, however, do not distinguish the discrepancies of the calculation of lost time and the delay computations among different compound left-turn protection.

In the experimental design of this study, special efforts have been made to address the major strengths and weaknesses of the previous studies. A large number of carefully designed simulations were performed to achieve this goal.

CHAPTER THREE

STUDY METHODOLOGY

This chapter presents a method for exploring detailed traffic information in NETSIM. The method uses the intermediate NETSIM output files which are produced for post-processor animation to get the necessary traffic data for model calibration. This method also can be used to analyze other complex traffic operations.

This chapter first describes the research approach. Then the methodology and data flow infrastructure are explained.

Research Approach

There are three basic techniques for studying traffic operations: field observation, mathematical analysis, and simulation. Although field studies provide direct information about traffic operations, they usually are time consuming and costly. Furthermore, it is difficult to generalize from field results because each traffic condition observed is essentially unique. Hence, it is generally desirable to formulate traffic operations as a model and use field observations for validation or calibration. Further, mathematical formulations of traffic operations under various flow conditions, geometric configurations, and signal phase sequences compounded with the stochastic nature of traffic becomes mathematically intractable. Computer simulation offers many of the advantages over the other two techniques.

Among the several traffic simulation models that are available, both the TEXAS model and NETSIM are excellent microscopic simulation models. NETSIM was chosen for this study for several reasons. One limitation of the TEXAS model is that it deals exclusively with one intersection (or two closely-spaced intersections) instead of a network. In urban areas, it is a common practice to coordinate traffic signals in a network; consequently, traffic conditions at one intersection may be affected by those at other nearby intersections.

Moreover, the TEXAS model starts simulations and data collection with an empty system; statistics collected during the initial transient period are unstable. On the other hand, NETSIM does not start data collection in a user-specified initial period or until a system reaching equilibrium.

More important, NETSIM allows the generation of a series of intermediate preprocessor data files. These files contain considerably detailed traffic data which are used intensively in this study.

The research approach was formed to accomplish the objectives of this study. A brief outline of the research approach follows.

1. Analyze existing models. Identify the similarities and differences among the existing analytical models and NETSIM. Investigate the critical variables important to left-turn flow analysis methodology.

2. Develop submodels for estimating critical variables. Using NETSIM, factors that construct various traffic conditions are studied to determine if these factors are significantly important to the critical variables. Then NETSIM is used to build the submodel for estimating these critical variables.

3. Validate proposed left-turn analysis methodology. NETSIM's strong capabilities for modeling almost any situation allow for the validation of the models developed above.

A more specific description of the research approach is illustrated in Figure 3.1. Left-turn analysis methodology can be classified into three areas by their complexities. These three areas are as described in the previous chapter: exclusive permitted left-turn movements, shared permitted left-turn movements, and compound left-turn protection. The model for estimating the saturation flow rate of exclusive permitted left-turn lanes is studied first since exclusive permitted left-turn lanes can be treated as a special case of shared permitted left-turn lanes by assuming no thru movement in the shared lanes. Then, based on the results of exclusive permitted left-turn study, the research can be extended to shared permitted left-turn movement by adding the critical variables only important to shared-lane operation. Finally, the research covers all possible left-turn operations.

Data Flow Structure

The output of NETSIM includes a variety of measures of effectiveness (i.e., speed, volume, density, delay, spillback, queuing, turn movements, fuel consumptions, and emissions) on each link of the network. It is impossible to get the information necessary for calibrating the left-turn saturation flow rate from the output report, such as the opposed left-turn saturation flow rates during the unsaturated green time and number of sneakers. Therefore, an alternative methodology is needed.

Recent enhancements in NETSIM allow for the generation of a series of intermediate preprocessor data files with the extension .F4n (n = 0 to 9). One of these files, .F41, contains

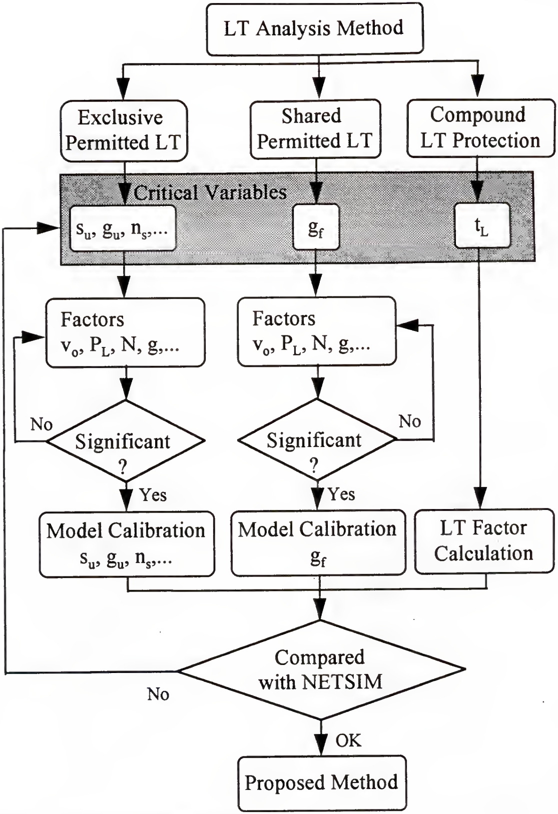


Figure 3.1. Flow chart of research approach.

the link, vehicle, and blockage data for animation. In every second, one record is written for each link in the network. Each link record may be followed by one or more records containing information about the vehicles existing on the link. When no vehicles exist on a given link, this vehicle data record is not written. These data are stored in binary format. A program, named cnvf41.exe, was developed to convert the binary codes of these files into readable ASCII codes.

The intersection configuration and traffic volume are coded into the Wizard of Helpful Intersection Control Hints (WHICH) (Courage and Wallace, 1994). From WHICH, the data can be passed to NETSIM for simulation. To make the NETSIM productive, both a preprocessor and postprocessor were developed by Courage and the author of this dissertation, respectively. The preprocessor, called RANGER, generates several NETSIM data sets with specified data varied systematically throughout a given range. This allows several NETSIM runs to be made to examine the effect of certain items of input data on the results. The postprocessor, cnvf41.exe, extracts the information needed for model calibration. Figure 3.2. shows the flow chart of data structure.

A step-by-step simulation procedure is given as follows:

Step 1. Set up a NETSIM input file for a standard intersection shown in Figure 3.3. All adjustment factors not given in the figure are assumed to be 1.0. In addition, set an oversaturated volume for the opposed left-turn traffic (so that capacity can be obtained).

Step 2. Run the simulation for an hour. Vary the opposing flow range from 10 to 1800 vph by increments of 10 vph.

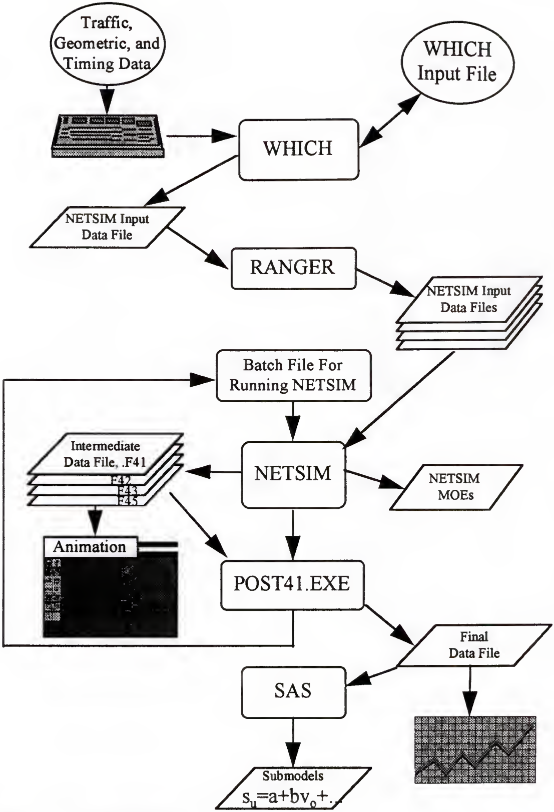


Figure 3.2. Flow chart of data structure.

Step 3. Convert the binary ".F45" files to ASCII ".F45" files.

Step 4. Extract the opposed left-turn lane capacity and the unsaturated green time (g_u)

Step 5. Compute the saturation flow rate using the following formula:

$$S'_u = c \times \frac{C}{g_u} \tag{3.1}$$

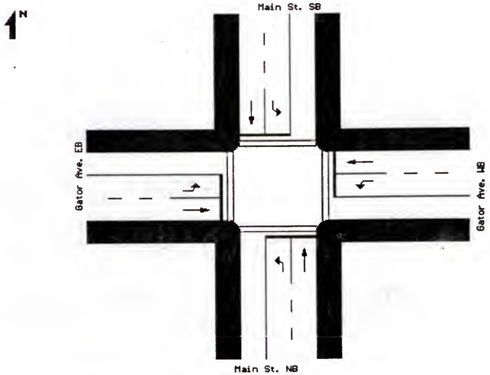


Figure 3.3. Standard intersection configuration.

CHAPTER FOUR

MODEL ELEMENTS

This chapter initially evaluates NETSIM in modeling basic intersection conditions (without the effect of left turns). Then three stages of left-turn studies are classified according to the different operational characteristics of left-turn lanes. The three stages are: (1) exclusive permitted left-turn lanes, (2) shared permitted left-turn lanes, and (3) compound left-turn protection. In each stage, the factors affecting left-turn saturation flow rate are initially investigated.

Preliminary Testing of the TRAF-NETSIM Model

In this section, an intensive evaluation is performed. The discussion focuses on two specific comparisons. The first reconciles the delay estimates from NETSIM and the HCM. The second part explores the differences between the NETSIM and the HCM methods for modeling traffic signal delay.

Reconciling Delay Estimates from NETSIM and HCM

In preparation for the comparing of outputs produced by the HCM against NETSIM, it is necessary to examine the exact definition of delay used by these two techniques and to reconcile any differences between them. Both methods produce estimates of a variable called "stopped delay per vehicle," but the basis for their computation is entirely different.

NETSIM defines stopped delay in terms of the accumulated number of seconds during which a vehicle is actually traveling at zero speed (i.e., "locked wheel" delay). The HCM, on the other hand, defines stopped delay in terms of time spent at a speed of less than 5 mph. It is computed analytically by dividing the total delay (defined previously) by a factor of 1.3. This factor was developed as the average ratio of total delay to stopped delay (McShane and Roess, 1990).

Furthermore, the definition of total delay is different between the HCM and NETSIM. In the HCM, total delay is the delay caused by a signal. NETSIM's total delay includes two parts. The first is cruise delay, which caused by the interaction of slower vehicles. The second is signal delay.

The various delay definitions can be seen in Figure 4.1, which shows a graphical representation of the time-space trajectory of an average vehicle delay as it passes through an intersection. Note that the vehicle approaches the intersection at a constant cruise speed and undergoes a period of deceleration before coming to a full stop. The stopped time is represented by the vertical segment of the trajectory since this segment portrays a time interval in which there is no change in the special position. At the end of the stopped-delay interval, the vehicle undergoes a period of acceleration until the cruise speed is reached. The various delays reported by the HCM and NETSIM models are indicated on the left and right sides of the intersection, respectively.

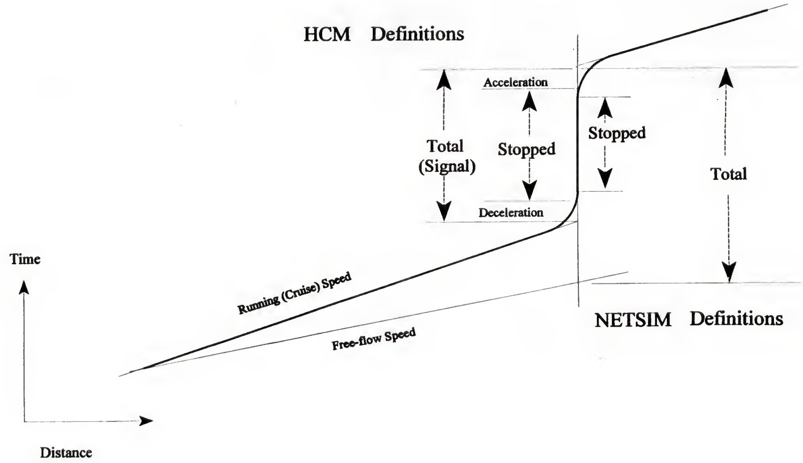


Figure 4.1. Comparison of delay definition.

In Figure 4.1, the HCM model considers the total delay to be the vertical distance (time) between the "cruise" lines for entering and leaving the intersection. This is the delay due to the signal. If there is no signal, the HCM total delay will be zero. However, the NETSIM's total delay is referenced to the free-flow speed of the approaching vehicles instead of the cruise speed used by the HCM. Free-flow speed is a value referred to as the highest cruise speed a vehicle can travel on a link. As the flow on the link increases, the cruise speed decreases. Specifically, the HCM total delay is the delay only caused by signal (i.e., signal delay), and the NETSIM total delay includes signal delay and cruise delay. Therefore, the NETSIM total delay is by definition higher than the HCM total delay because it includes the "cruise delay."

A procedure was developed to show how signal delay (HCM's total delay) can be determined using the NETSIM model. The procedure requires two steps. The first considers the relationship between input free-flow speed, cruise speed, and volume to saturation flow rate (v/s) ratios. The second constructs a model to differentiate signal delay from NETSIM total delay.

Speed-flow Analysis

The study methodology involved a comparison of the input free-flow speed with the modeled average cruise speed at midblock for various v/s ratios. The general relationship is

$$W = a \times S_c \times (S_f - S_c) \tag{4.1}$$

and

$$S_c = \frac{S_f}{2} \pm \frac{\sqrt{a^2 S_f^2 - 4aW}}{2a} \tag{4.2}$$

where W = v/s ratio,

S_f = input free-flow speed,

S_c = cruise speed, and

a = model parameter.

The intersection being studied consists of one lane in each direction. Only thru vehicles are considered in this analysis, i.e., no right- or left-turning vehicles. The link length used was 2640 feet. The phase split was 50/50. A saturation headway of 2 seconds per vehicle (saturation flow rate of 1800 vphpl) and cycle lengths of 60 and 90 seconds were used in this analysis. These values reflect an ideal intersection setup with long link lengths and acceptable real-life values for green times, cycle lengths, and saturation flow rates.

Using the procedure described earlier in Figure 3.1, a time-space diagram with an approach volume of 500 vph is plotted in Figure 4.2. Every point reflects the location and time of a vehicle traveling on the link every second. The average cruise speed can be obtained by taking the average slope of these trajectories. In this example, the cruise speed is 39.3 feet per second, which is slower than the free-flow speed (44 feet per second). The slower cruise speed is due to the fact that NETSIM considers speeds from 75 to 127 percent of the input speed and assigns them to each vehicle randomly. A low-speed vehicle would tend to reduce the speeds of all the following vehicles.

By this process, a database was created containing the NETSIM cruise speeds, free-flow speed, and v/s ratios. A total of 192 simulation runs were carried out to create the database. The relationship of cruise speed and v/c ratios for the free-flow speed of 30 mph (44 ft/sec) is plotted in Figure 4.3.

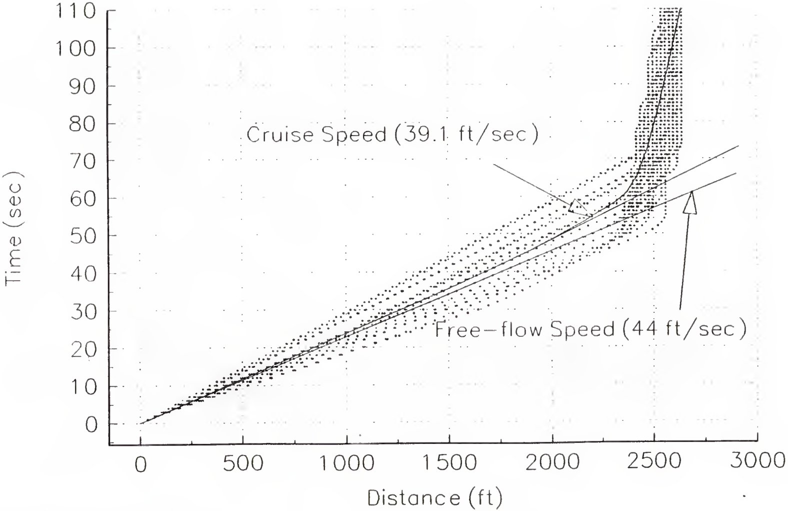


Figure 4.2. Time-space diagram.

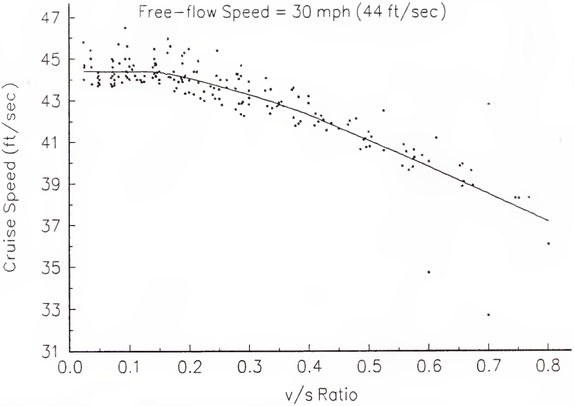


Figure 4.3. Relationship of v/s ratio and cruise speed.

A relationship between cruise speed, free-flow speed, and v/c ratios is sought using regression analysis for all data. The model parameter, a, in Equations (4.1) and (4.2) is found to be 0.002295. The coefficient of determination, R-square, was estimated to be 0.93. This means that the equation fits the data reasonably well. The p-value of the estimated coefficient is less than 0.05, indicating that it is significant at the 95 percent confidence level.

Signal Delay in NETSIM

By definition, the NETSIM total delay is equal to the travel time at cruise speed plus intersection signal delay minus travel time at free-flow speed. Thus, signal delay is equal to total delay minus travel time at cruise speed plus travel time at free-flow speed. This relationship can be represented by the following equation:

$$SD_N = D_N - \frac{L}{S_c} + \frac{L}{S_f} \tag{4.3}$$

where SD_N = NETSIM signal delay (sec/veh),

D_N = NETSIM total delay (sec/veh),

L = Link length (ft), and

S_c and S_f were defined previously.

Using the 192 data sets in the speed-flow analysis, the relative accuracy of the recommended model was compared with the 1994 HCM model. The results of the comparison are shown in Figure 4.4. The results indicate that the HCM total delays are higher than those from the NETSIM. This fact also can be demonstrated using the queue accumulated polygon method in the following section.

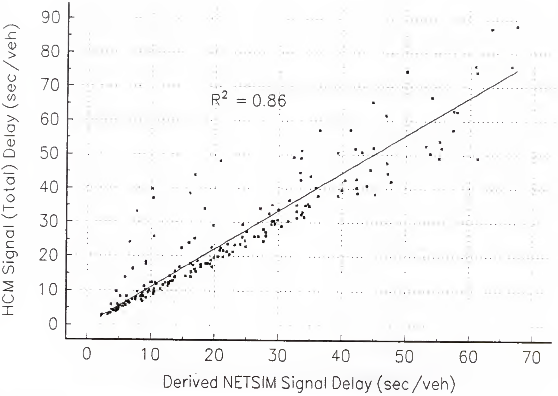


Figure 4.4. Comparison of simulated and HCM signal delays.

Traffic Signal Delay Model

The traditional delay formulation used by virtually all analytical models is based on two components, or terms, which are added together to produce the computed delay per vehicle. The first is uniform delay, which refers to the average vehicle delay experienced assuming that traffic demand is the same for all signal cycles. The second is incremental delay, which occurs because individual cycle failure and the random arrivals of vehicles.

The first delay component D_1 is given by

$$D_1 = \frac{C(1-\frac{g}{C})^2}{2(1-\frac{g}{C}\times X)} \tag{4.5}$$

where X = volume-to-capacity ratio.

This formula is valid when the arrival rate is uniform throughout the cycle. It is used in the British and Australian delay calculation methods. Equation (4.5) is also the basis for delay calculation in the HCM, with two important modifications. The first uses stopped delay rather than total delay as a measure of performance for signalized intersections. Second, stopped delay is defined not in terms of locked wheel delay (i.e., totally stopped) but in terms of time spent at less than a certain speed, such as 5 mph (McShane and Roess, 1990). Total delay also includes the time lost because of deceleration and slower movement through the intersection. According to the HCM, the total delay is 1.3 times the stopped delay.

The incremental delay is called the random-plus-saturation delay because of random arrivals and individual cycle failures in the HCM. Akçelik (1988) proposes a generalized time-dependent expression in the form

$$D_2 = 900 \times T \times X^{n} \times [X-1+\sqrt{(X-1)^2 + \frac{m(X-X_o)}{cT}}] \quad (4.6)$$

where X_o = volume-to-capacity ratio below which overflow delay is negligible, and

$X_o = a + bsg$

where s = saturation flow rate (vph),

g = effective green, and

a, b, m, n = parameters.

The delay model parameters a, b, m, and n depend on the distribution of arrivals and departures. Model parameters as presented by Akçelik (1988) are given in Table 4.1.

Table 4.1. Calibration parameters for d_2 in analytical models.

Method	Model parameters			
	n	m	a	b
1985 HCM	2	4	0	0
Australian	0	12	0.67	1/600

Source: Akçelik (1988).

To demonstrate NETSIM modeling the two delay components, several runs were made in which traffic streams with different flow rates were introduced into links with different lengths. The results are shown in Figures 4.5 and 4.6 for single-lane and multilane links, respectively. Volume-to-capacity ratios from 0.6 to 1.0 and link lengths of 150, 500, 1,000, 1,500, and 2,000 feet are represented in these figures.

Figures 4.5 and 4.6 illustrate that the delay of the 150 foot link is close to the analytical uniform delay. This is because the link length is so short that the cycle failure delay can not occur. Moreover, 100 percent of the free-flow speed was assigned to every type of driver so that the traffic would not be affected by slower vehicles.

To compare the delay for link lengths of 500 to 2,000 feet with the existing models, NETSIM underestimates the delay values. For overflow delay, NETSIM predicts lower values when the v/c ratio nears 1.0. This is due to the fact that NETSIM generates less random fluctuation in the arrival flow rate than the existing models do. This can be illustrated by increasing the range of free-flow speed percentage. Table 4.2. lists the values assumed for different profiles of free-flow speed percentage. In this table, P_n (n=0-4) represents different profiles of free-flow speed pertantage. Simulations with these profiles were performed and again compared with the HCM total delay. The result is shown in Figure 4.7. As seen in the figure, the total delay of NETSIM approaches the curve of the HCM total delay as the randomness of traffic increased.

On the other hand, the default values of the free-speed percentage profile can be approached by changing the parameter value of the HCM overflow delay equation. Figure 4.8 shows a comparison of the default NETSIM setting with different HCM m values. As seen in the figure, the default value in NETSIM is between m=4 and m=8.

Table 4.2. Profiles of free-flow speed percentage.

Driver Type	Profile of free-flow speed Percentage				
	P_0 (default)	P_1	P_2	P_3	P_4
1	75	70	65	60	54
2	81	77	72	67	63
3	91	87	83	78	72
4	94	90	87	82	81
5	97	93	91	89	86
6	100	101	102	104	106
7	107	112	116	119	123
8	111	116	120	126	130
9	117	122	127	133	138
10	127	132	137	142	147

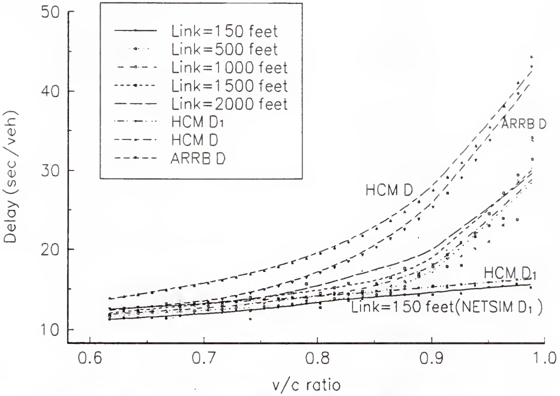


Figure 4.5. Comparison of NETSIM's delay with existing models for single-lane approach.

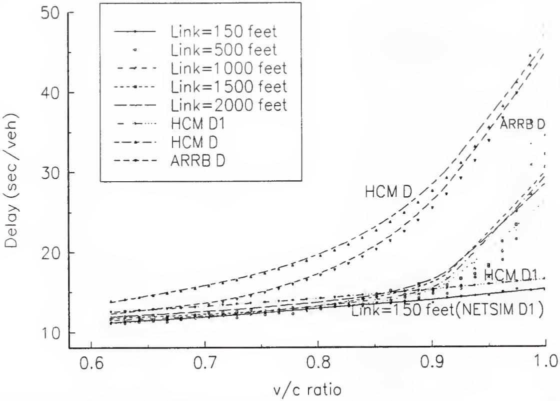


Figure 4.6. Comparison of NETSIM's delay with existing models for multilane approach.

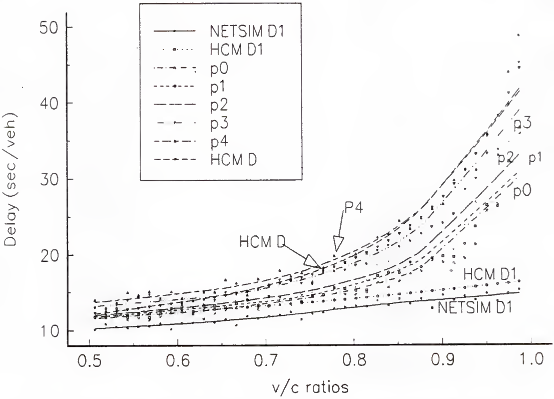


Figure 4.7. Comparison of HCM total delay with NETSIM total delay for different profiles of free-flow speed percentage.

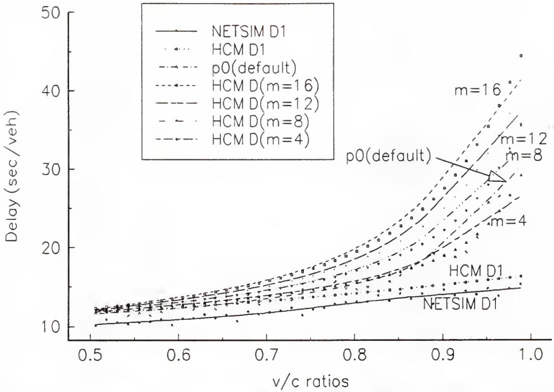


Figure 4.8. Comparison of NETSIM default setting with different HCM m-values.

Saturation Flow Rate of Exclusive Permitted Left-turn Lanes

The literature on modeling of the saturation flow rate of exclusive permitted left-turn lanes suggests that there are three components of departures during the green phase. They are (1) jumpers, (2) the opposed left-turn saturation flow rate (s_u) during the unsaturated green time (g_u) and (3) sneakers (n_s). Jumpers refer to those leading left-turn vehicles in various cycles that turn in front of the leading vehicle shortly after green onset. Because the number of jumpers are usually small, most existing models omit this term.

Both the 1985 HCM and the ARRB models use the first term to represent the opposed turn saturation flow rate during the unsaturated green time (g_u) and the second term to indicate the number of sneakers. The 1985 HCM model assumes two sneakers per cycle. The ARRB model allows users to specify the number of sneakers (n_s). Sneakers are treated implicitly in the 1994 HCM by imposing a lower limit of two vehicles per cycle on the capacity of each exclusive left-turn lane with permitted movements. In brief, the saturation flow rate of permitted exclusive left-turn lanes depends primarily on the opposed left-turn saturation flow rate, unsaturated green time, and number of sneakers.

Before using a simulation model to study left-turn performance, it is desirable to know how well the model simulates traffic in the real world and to critically evaluate the features that pertain to left-turn operations. Moreover, in order to draw appropriate conclusions from simulation results, the basic traffic and geometric conditions under which simulation studies are carried out have to be specified. In this section, specifications for simulation runs will be outlined, and factors important to left-turn operations such as opposing flow rate, number of opposing lanes, green time, and cycle length will be examined.

Simulating Opposed Left-turn Saturation Flow Rate

The relationship between the opposed left-turn saturation flow and the opposing flow rate is shown in Figure 4.9. A smoothing factor of 0.5 has been applied to the curve. When compared to the existing analytical models, NETSIM clearly overestimates the opposed left-turn saturation flow rate s_u, especially when the opposing flow rate is nearly saturated. This is because NETSIM includes a non-changeable default value for both the sneakers and jumpers. Therefore, the following steps are needed to remove the sneakers and jumpers to compute the saturation flow rate based on the opposed left turns only.

Step 1: Remove Sneakers. The high opposed left-turn saturation flow rate that occurs when the opposing flow nears the saturated condition is due to the use of a constant number of sneakers for a short unsaturated green time. This problem can be solved by removing the sneakers, which is done by excluding from the ".F45" file those vehicles making left turns during the intergreen intervals. Figure 4.10 shows the opposed left-turn saturation flow rate without sneakers.

Step 2: Remove Jumpers. A left-turn jumper is a vehicle that is first in the queue when the signal changes to green; the jumper executes the left turn maneuver immediately before the on-coming queues can discharge. In NETSIM, the default value is 38 percent which is too high for most signalized intersections. The aim was to remove the jumpers from the simulation. The same set of output without jumpers was plotted in Figure 4.11.

Upon removing sneakers and jumpers, the relationship of the opposed left-turn saturation flow rate (s_u) and the opposing traffic flow rate (v_o) can be obtained. Then, the left-turn equivalence, which is used in the ARRB and the HCM, also can be determined as :

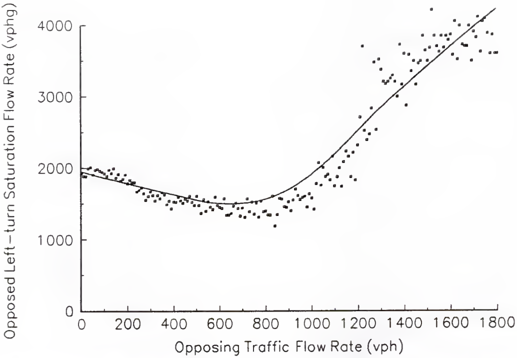


Figure 4.9. Simulated opposed turn saturation flow rate with sneakers and jumpers.

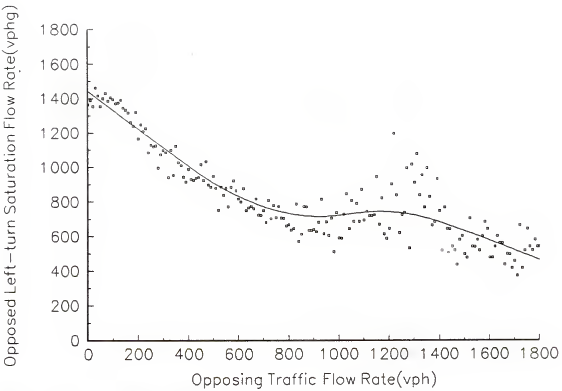


Figure 4.10. Simulated opposed saturation flow rate with jumpers.

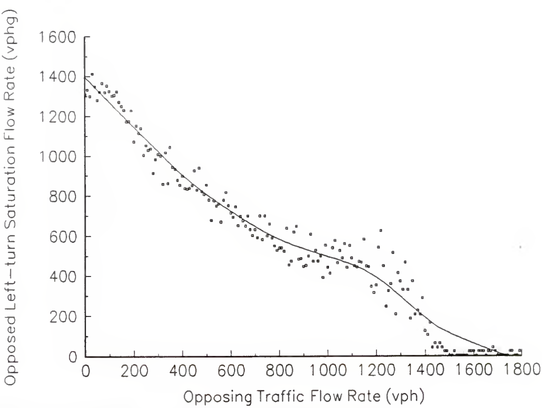


Figure 4.11. Simulated opposed left-turn saturation flow rate.

$$E_{L1} = \frac{s}{s_u} \tag{4.7}$$

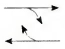
Effect of total number of signal phases. As shown in Table 2.1, the 1994 HCM considers the total number of signal phases as an important factor for estimating the opposed left-turn saturation flow rate. However, by comparing two examples, it can be shown that the total number of signal phases should not be a factor for determining s_u. The traffic conditions and the opposed left-turn saturation flow rate for the two examples are shown in Table 4.3. These two examples are identical with one exception: the second example contains an additional protected left-turn phase in the east-west direction. After analysis of these two examples using the 1994 HCM procedures, the results show that there are significant differences in the saturation flow rate for the northbound and southbound left-turn lanes. The reason for this deviation is because a different E_{L1} value is used for a different total number of signal phases. Obviously, this is counterintuitive since the traffic conditions for the northbound and southbound approaches of these two examples are identical.

Unsaturated Green Time

The value of g_u , the unsaturated portion of green after clearance of the opposing queue, is expected to be strongly related to the volume of opposing flow. Following similar procedures, a plot of unsaturated green time, g_u , is displayed in Figure 4.12. However, the g_u is not related to the existence of sneakers and jumpers.

The trend displayed by the figure is logical. As the opposing flow increases, the value of g_u decreases as expected. As the opposing flow approaches to the capacity of this lane group, 1600 vph, the value of g_u reduces to zero. This reflects the fact that there is no

Table 4.3. Comparison of opposed left-turn saturation flow rate.

Example	Phase			Movement		No.of Lanes	Flow Rate	E_{L1}	s_u	f_{LT}	s	c	v/c
1	1		G:37sec Y:3sec	EB	LT	1	100	3.65	520	.17	323	149	.67
					TH	1	500						
				WB	LT	1	100	3.65	520	.17	323	149	.67
					TH	1	500						
	2		G:37sec Y:3sec	NB	LT	1	100	3.65	520	.17	323	149	.67
					TH	1	500						
				SB	LT	1	100	3.65	520	.17	323	149	.67
					TH	1	500						
2	1		G:7sec Y:3sec	EB	LT	1	100	5.75	330	.13	253	253	.395
					TH	1	500						
	2		G:27sec Y:3sec	WB	LT	1	100	5.75	330	.17	253	253	.395
					TH	1	500						
	3		G:37sec Y:3sec	NB	LT	1	100	5.75	330	.11	210	97	1.03
					TH	1	500						
				SB	LT	1	100	5.75	330	.11	210	97	1.03
					TH	1	500						

unsaturated green available when the opposing flow is saturated. This also suggests that the volume-to-capacity ratio (v/c) of the opposing flow should be used, instead of the opposing flow (v_o), for determining the value of g_u.

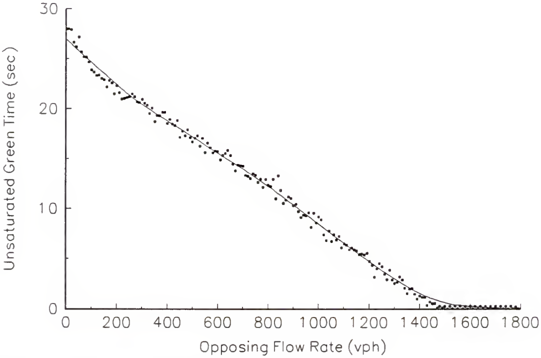


Figure 4.12. Simulated unsaturated green time.

Number of Sneakers

Sneakers are permitted left-turn vehicles that exit the intersection at the end of the green phase, usually during the intergreen interval. It is common to assume that two vehicles per cycle may be released from a queue in this manner. The 1985 HCM adds two sneakers per cycle in the saturation flow rate. The ARRB and TRANSYT-7F allow users to define the number of sneakers. However, sneakers are treated implicitly in the 1994 HCM by imposing a lower limit of two vehicles per cycle on the capacity of each exclusive left-turn lane with permitted movements. For purposes of this research, the number of sneakers was investigated.

Following the procedures presented in Chapter Three, a plot of the number of sneakers (n_s) was determined. This plot is shown in Figure 4.13. However, n_s is a function not only of the opposing flow but also of the opposed flow. The volume of the opposed flow was varied from 50 to 1000 vph by increments of 50 vph.

As seen in the figure, the number of sneakers decreases as the opposing flow rate approaches 1400 vph and then increases as the opposing flow increases. This trend reveals that given an opposed-flow rate, the number of sneakers is affected not only by the opposing flow but also by the opposed turn saturation flow rate (s_u). In other words, the number of sneakers decreases as the opposing flow increases due to the decrease of acceptable gaps. The number of sneakers increases because of the decrease of the opposed left-turn saturation flow rate. In this situation, the left turners can not execute the turn during the green time and wait at the stop line until the signal turns to yellow. All left turners at this time are treated as sneakers.

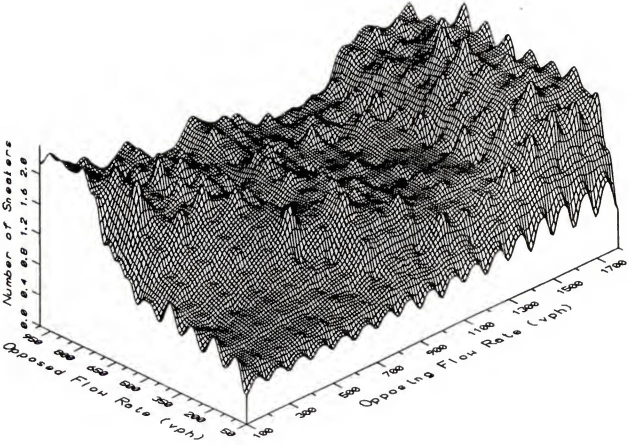


Figure 4.13. Number of sneakers in various traffic conditions.

Taking the maximum opposed flow rate, the relationship between the maximum number of sneakers and the opposing flow rate can be obtained. This relationship is shown in Figure 4.14.

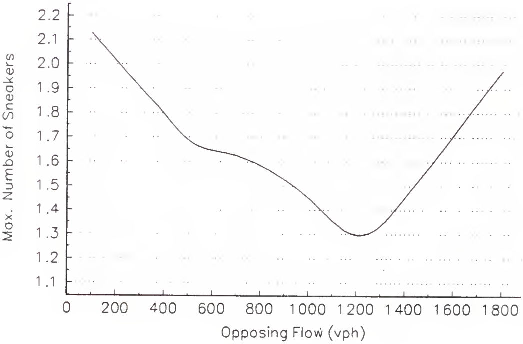


Figure 4.14. Maximum number of sneakers as a function of opposing flow rate.

Saturation Flow Rate of Shared Left-Turn Lanes

Left-turn analysis becomes extremely complex when permitted left turns are made from shared lanes. Consider a vehicle in a shared lane wishing to turn left against an opposing vehicle flow. There are several distinct portions of the green phase during which left turns face dramatically different operating conditions.

When the green is initiated, an opposing queue of vehicles begins to move. It takes this opposing queue of vehicles g_q seconds to clear the intersection. During this time, there are no gaps in the opposing traffic stream, and no left turns may be made. Under extreme congestion, $g_q = g$, and the left turns only occur at the end of the phase during the clearance interval. If g_q is less than g, the remainder of the phase is not affected by the clearance of an opposing queue. This is called the "unsaturated" green time and is given the symbol g_u. During this portion of the green, left-turn vehicles must select gaps through an unsaturated opposing flow. Thus, the delaying effect of the opposing flow on left-turning vehicles has two components: the length of g_q, during which no left turns can be made, and the intensity of the opposing flow during g_u.

If left turns are being made out of a shared lane, another factor intervenes: the clearing of the opposing queue does not affect the flow in the shared lane until the first left-turning vehicle arrives. The time until the arrival of the first left-turning vehicle in the shared lane is designated g_f. During this time, left turns have no impact on flow in the shared lane. The existing models for the left-turn adjustment factor also assume that left turns have no impact on the operation of the shared lane during g_f, before the first-turning vehicle arrives (the effective left-turn factor during this period is 1.00). During the period g_q-g_f, that is, the green time between arrival of the first left turn and the clearance of the opposing queue, the shared lane is assumed to be blocked by the waiting left turner (the effective factor during this period is assumed to be zero). During the unsaturated portion of the green, left turns are made through an unsaturated opposing flow.

When permitted phasing is used in conjunction with an exclusive left-turn lane, the same discussion holds, except that $g_f = 0$. The first left-turning vehicle is assumed to be present as soon as the green signal is initiated. This situation is identical to the condition discussed in previous section of this paper. Therefore, in determining the saturation flow rate of shared lanes, the key variable is the value of g_f. In this section factors important to the value of g_f such as green time, cycle length, left-turn flow rate, and proportion of left turns are examined.

The Existing Models

From the literature, the value of g_f is expected to be strongly related to green time and the number or proportion of left turns being made. There are three existing analytical models for estimating the time before the first left turner arrives. These models are the 1985 HCM model, the ARRB model, and the 1994 HCM model.

The 1985 HCM model. In the 1985 HCM, the duration of the green phase during which thru vehicles may move in a shared lane until a left-turning vehicle arrives is estimated as

$$g_f = \frac{2P_T}{P_L} \times [1 - P_T{}^{0.5 g_q}] \qquad (4.8)$$

where $g_q = g - g_u,$

P_L = proportion of left turns in shared lane, and

P_T = proportion of thru vehicles in the shared lane.

The ARRB Model. Akçelik (1988) introduced the concept of the "lane interaction" method for calculating the saturation flow rate of a shared lane at signalized intersections. This method treats the intervals of lane blockage in shared lanes as effective red times. First, the "blocked intervals" (the first and third interval), during which one movement receives a red signal (blocking movement) and the other receives a green signal (blocked movement) in the shared lane, are calculated. Second, the "common green interval" (the second interval), during which both movements receive a green signal, is calculated. Then, the number of departures by one movement before being blocked by the other (s_d) for each blocked interval is calculated using the following formula:

$$S_d = p_b^{F+1} \sum_{i=1}^{M-1} \frac{p_d^{i}(i+F)!}{F!(i-1)!} + M p_d^{M} \sum_{j=1}^{F+1} \frac{p_b^{F+1-j}(M+F-j)!}{(M-1)!(F+1-j)!} \tag{4.9}$$

where P_b = proportion of blocking flow,

P_d = proportion of blocked flow ($1-P_b$),

F = free queue parameter, and

M = maximum blocked departure (the product of the duration of the blocked interval g_d and the saturation flow s of the blocked movement).

The 1994 HCM. In the 1994 HCM, the duration of the green phase during which thru vehicles may move in a shared lane until a left-turning vehicle arrives is estimated as

$$g_f = \begin{cases} G\ \exp(-0.860LTC^{0.629})-t_L & \text{single-lane approaches} \\ G\ \exp(-0.882LTC^{0.717})-t_L & \text{multilane approaches} \end{cases} \quad (4.9)$$

here G = actual green time for the phase,

LTC = average number of left turns per cycle, and

t_L = total lost time per phase (second).

All of the three models consist of different modeling structures. The 1985 HCM considers the proportion of left turns and the saturated green time of the opposing flow as the critical parameters. The 1994 HCM uses the green time and average number of left turns per cycle. The ARRB model allows the modeling of a "free queue." A free queue is the number of vehicles that must wait in queue to make a permitted turn before the lane is completely blocked to thru traffic. A left turner waiting for gaps to proceed can often move to the center of the intersection and still allow through vehicles to move freely. This situation is especially important in modeling single-lane approaches where the thru vehicles will try to, and often succeed, in moving despite a left-turn vehicle waiting in front of it. Both the 1985 HCM and 1994 HCM ignore the free-queue effect; thus, the first left turner is assumed to block the entire lane. Table 4.4 lists the factors used in the three different models.

Despite the fact that all of the parameters listed in Table 4.3 are intuitively critical, it was necessary to study the effect of these parameters in the value of g_f in basic cases before designing the experiments for model calibration.

Table 4.4. Parameters employed in the existing models.

Model	Factors
1985 HCM	1. proportion of left turns 2. saturated green time of the oppsoing flow
ARRB	1. proportion of left turns 2. saturated green time of the opposing flow 3. number of free queue
1994 HCM	1. number of approach lanes 2. number of left turns per cycle 3. actual green time

Effect of green time

Estimations of the average g_f value under differnt left-turn volumes and green times are shown in Figure 4.15. The relationship is in the form of simple logic. When there are no left turns, g_f is equal to the effective green time. As the left turns increase, g_f asymptotically approaches zero. Moreover, this negative-exponential relationship is similar to the 1994 HCM model, which was calibrated using an empirical database.

Effect of proportion of left turns

The 1985 HCM uses the proportion of left turns (P_L) as a major factor in predicting the g_f value, while the 1994 HCM ignores the proportion of left turns. Estimations of average g_f under different proportions of left turns and approach volumes (including left-turn and thru movements) are plotted in Figure 4.16. As shown in the figure, given a specific proportion of left turns, as the approach volume increases, the value of g_f decreases. When the proportion of left turns increases, the value of g_f decreases rapidly. However, the g_f value is different under a left-turn volume with different P_L. For example, Point A consists of a 1000 vph approach volume, 10 percent of left turns, and 20 seconds of g_f. Therefore, the left-turn volume on Point A is equal to 100 vph. On the other hand, Point B consists of a 100 vph approach volume and the left-turn volume is 100 vph as well. However, the g_f is about 12 seconds. Thus, the proportion of left turns has a significant effect on g_f.

Effect of free-queue maneuver

The free-queue vehicles are the thru vehicles, in single shared lanes, that pass the blocked left-turn vehicle and exit the intersection. This maneuver can not be simulated in NETSIM. However, an alternative can be used to overcome this problem by measuring g_f until the second left-turn vehicle arrives at the stop line. A comparison of g_f with and without free-queue maneuvers is presented in Figure 4.17.

Assignment of Lost Time

In the analysis of the permitted portion of compound left-turn protection, the basic models described previously may be applied. The difficulty is in selecting values of G, g, g_f, g_q, and g_u for use in these models. The equation for g_f is indexed to the beginning of the effective green in the subjective green for the opposing flow. When compound left-turn protection exists, these equations must be modified to account for shifts in the initiation and overlap of various green times. Generally speaking, compound left-turn protection includes two different phasing operations: protected-plus-permitted (leading) left-turn phasing and permitted-plus-protected (lagging) left-turn phasing. Each of these cases was analyzed separately.

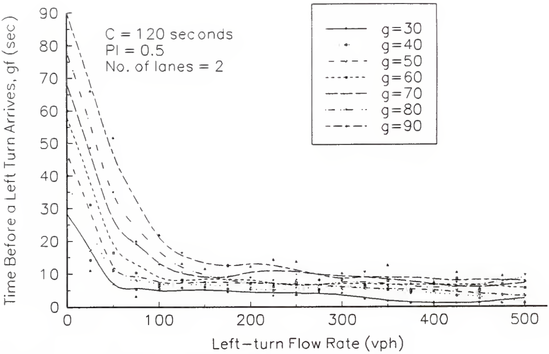


Figure 4.15. Effect of effective green on g_f.

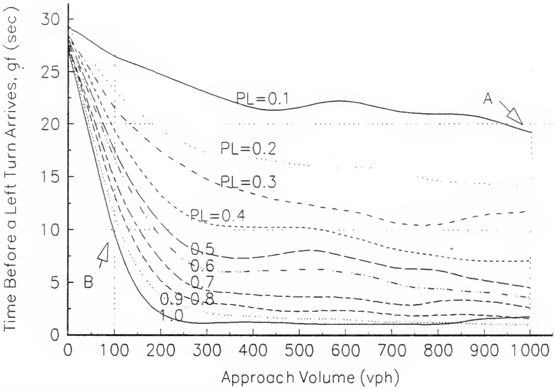


Figure 4.16. Effect of proportion of left turns on g_f.

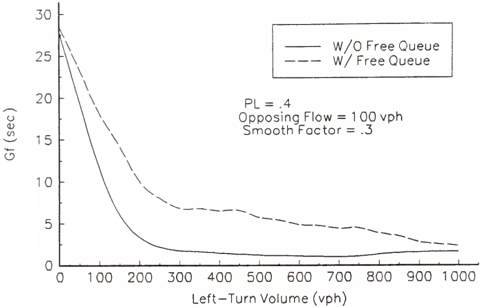


Figure 4.17. Effect of free-queue maneuver on g_f.

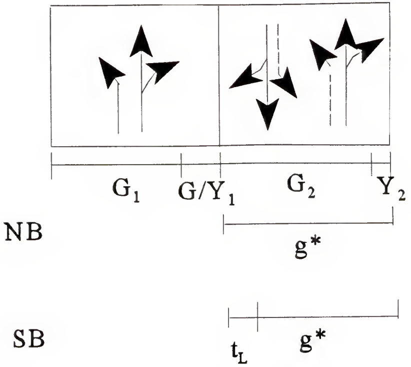


Figure 4.18. Green time adjustments for protected-plus-permitted phasing.

The diagram of protected-plus-permitted phasing is shown in Figure 4.18. The effective green time for the permitted phase, g’, is equal to G_2+Y_2 for the northbound direction and G_2+Y_2 - t_L for southbound direction. Note that there is no lost time for northbound movements since both were initiated in the leading phase and the lost time is assessed there. Figure 4.19 illustrates this phenomenon by using NETSIM. The northbound vehicles continuously discharge from the stop line without any start-up lost time during the G/Y_1 interval. On the other hand, for the permitted-plus-protected phase, the start-up lost time should be applied for northbound and southbound vehicles since both were initiated in the permitted phase.

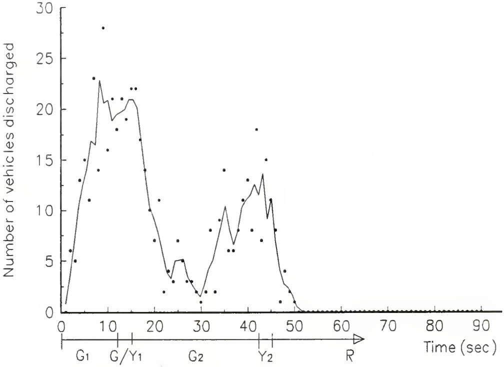


Figure 4.19. Allocation of lost time in NETSIM.

CHAPTER FIVE

PROPOSED MODEL FOR ESTIMATING SATURATION FLOW RATE OF PERMITTED LEFT-TURN LANES

This chapter presents a model for estimating the saturation flow rate of left-turn lanes at signalized intersections. This model was calibrated from extensive simulation results. This model not only is capable of realistically incorporating related critical factors, but also possesses the mathematical logic of the existing analytical models. This model will be referred to as the proposed model.

This chapter first presents the proposed model for estimating the saturation flow rate of permitted left-turn lanes. Second, the saturation flow rate for the left-turn lanes with compound left-turn protection is discussed. Finally, the proposed model is incorporated into the HCM Chapter 9 procedure so that the model can cover all of the possible left-turn operations.

Model Development

Generally speaking, exclusive permitted left-turn saturation flow rate depends primarily on the opposing flow rate, the number of the opposing lanes, and the signal timing. Any model that fails to take all of these factors into account can not yield consistently reasonable estimates for a left-turn saturation flow rate. This section presents the models consisting of these factors using the simulation results.

In order to make the experimental design feasible, unless otherwise specified, input variables were set to the following values:

(1) geometric configuration variables
  (a) lane width: 12 ft
  (b) inbound approach length: 2640 ft
  (c) outbound approach length: 2640 ft
  (d) angle of intersection: right angle
  (e) type of terrain: level
(2) driver characteristics variables (see Table 5.1)
(3) traffic variables
  (a) mean queue discharge headway: 2 sec
  (b) mean start-up lost time: 2.5 sec
  (c) desired free-flow speed: 30 mph
  (d) percentage of right turn: zero
  (e) pedestrian constraint: no
  (f) parking maneuver: no.

For exclusive permitted phasing, the following variables were investigated in the simulation experiments:

(1) number of opposing lanes: 1 - 3
(2) opposing traffic volume: 0 - 1900 vphpl
(3) cycle length: 60 -150 sec
(4) g/C ratio: 0.2 - 0.8.

Table 5.1. The input values applied to driver type.

Input values		Driver type									
		1	2	3	4	5	6	7	8	9	10
Free-flow speed (%)		76	91	91	94	97	100	107	111	117	127
Left-turn gap (sec)		7.8	6.0	6.0	5.4	4.2	4.5	4.2	3.9	3.6	2.7
Start-up lost time (%)	Link type 1	218	140	125	118	102	86	76	63	47	23
	Link type 2	258	140	143	118	95	76	57	38	29	0
Discharge headway (%)	Link type 1	170	120	120	118	180	110	90	70	70	50
	Link type 2	180	140	120	140	180	90	90	70	60	50

Based on the results of the simulation experiments, models were calibrated for estimating the exclusive permitted left-turn saturation flow rate:

The model for estimating the opposed left-turn saturation flow rate is as follows:

a. opposed by single-lane approach

$$s_u = 1615 - 1.281 \times v_o + 0.001331 \times v_o^2 - 0.000000635 \times v_o^3 \qquad (5.1)$$
$$(384 \ \textit{observations}, \quad R^2 = 0.98)$$

b. opposed by multilane approach

$$s_u = 1535 - 2.11 \times v_o + 0.001806 \times v_o^2 - 0.000000607 \times v_o^3 \qquad (5.2)$$
$$(384 \ \textit{observations}, \quad R^2 = 0.98)$$

The model for unsaturated green time

$$g_u = \frac{g_o - Y_o \times C}{2\beta - Y_o \times R_{PO}} \qquad (5.3)$$

where β = average departure rate (veh/sec), and is computed as

$$\beta = 0.0018 \times \frac{v_o}{\sqrt{g_o}} + 0.5 \qquad (5.4)$$
$$(543 \ \textit{observations}, \ R^2 = 0.93)$$

The model for estimating the maximum number of sneakers

$$n_s = 2.34 - 0.00174 \times v_o + 8.44 \times 10^{-7} \times v_o^2 \qquad (5.5)$$

Several statistical tests were conducted on these new models. The results provide the P- values to determine the significance of the coefficients and also give the R^2 value for an estimate of overall fit of the model. The P-values of all the estimated coefficients are less than 0.05, indicating that they are significant at the 95 percent confidence level. The R^2 values for the S'_u, g_u, and n_s are 0.98, 0.93 and 0.91, respectively. This indicates that the models are significant enough to be able to predict the dependent variables within the required degrees of accuracy.

Based on the investigation in Chapter Four, the significance of the parameters on the g_f value is clearly demonstrated. Regression models were calibrated for various traffic conditions:

for single lane approach

a. for free queue = 0

$$g_f = G' \times \exp(-0.4544 LTC^{0.9946} P_L^{-0.2383}) - t_L \tag{5.6}$$

b. for free queue = 1

$$g_f = G' \times \exp(0.14889 LTC^{1.3027}) - t_L \tag{5.7}$$

c. for free queue = 2

$$g_f = G' \times \exp(0.09639 LTC^{1.3045}) - t_L \tag{5.8}$$

for multilane approach

$$g_f = G' \times \exp(-0.3782 LTC^{0.8086} P_L^{0.1943}) - t_L \tag{5.9}$$

where G' = actual green time plus yellow and all red (i.e. $G' = g + Y + AR$),

LTC = the number of left turns per cycle,

P_L = proportion of left turns, and

g_q and t_L were defined previously.

This model has a good correlation coefficient and expresses a logical relationship. When there are no left turns, LTC= 0 and $g_f = g$. As LTC increases, g_f asymptotically approaches zero.

Relative to the existing models, the proposed model is similar to the 1994 HCM model. They have the same model structure.

Model Comparisons

In order to draw any appropriate conclusions in the proposed model, factors important to left-turn operation must be compared with previous research results. This section presents the comparison of the proposed model with the previous research results.

Opposed left-turn saturation flow rate. Figure 5.1 compares the proposed model for estimating the opposed left-turn saturation flow rate with the existing analytical models. When the opposing flow is less than 800 vph, all models provide a similar prediction. However, the 1985 HCM model estimates that the opposed flow is equal to zero when the opposing flow is over 1400 vph. This does not happen under real traffic conditions. The opposed saturation flow is supposed to approach zero as the opposing flow is nearly saturated. Compared to the proposed model, the ARRB model and the 1994 HCM model overestimate the opposed saturation flow rate and do not converge to zero for a near saturated condition

Left-turn headway. If an opposing gap is large enough to accommodate more than one left turn, the headway between successive left-turn vehicles has a significant effect upon how many left turns can go through it. The TEXAS Model suggests an average left-turn headway of 3.6 seconds. Fambro and Messer (1977) observed left-turn headways as follows: (1) permitted shared left-turn lane, 2.6 seconds; (2) permitted exclusive left-turn lane, 2.5 seconds; and (3) protected exclusive left-turn lanes, 2.1 seconds. Regression models on the outputs of NETSIM show an average headway between successive vehicles on exclusive

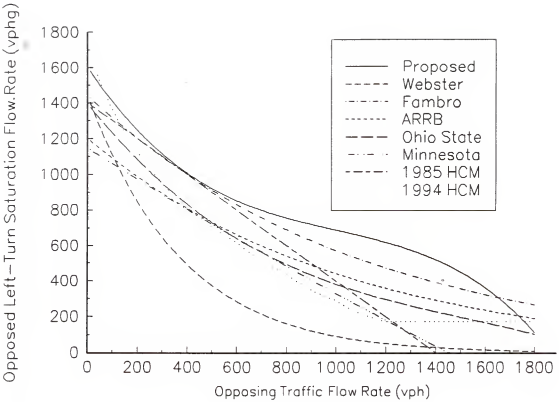


Figure 5.1. Comparison of proposed model for estimating opposed left-turn saturation flow rate with existing models.

permitted left-turn lanes as follows: (1) for opposed by single-lane approach, 3600/1615 = 2.23 seconds; and (2) for opposed by multilane approach, 3600/1535 = 2.35 seconds.

Gap-acceptance criteria. Gap acceptance criteria are often characterized by the critical gap, which is a gap of duration just sufficient for left turners to have an equal probability of accepting or rejecting it. Agent (1976) found a critical gap of 4.2 seconds at signalized intersections with a permitted exclusive left-turn lane. Other values of critical gap ranging from 3.8 to 5.8 seconds are reported in the literature. An average critical gap of 4.95 seconds is the default value in NETSIM. A plot of different critical gaps in NETSIM is shown in Figure 5.2.

Unsaturated green time. Figure 5.3 reveals a noticeable difference in determining the unsaturated green time. The 1994 HCM model predicts that there is no unsaturated green time once the opposing flow is over 1000 vph. Compared to the proposed model, the ARRB model and 1985 model overestimate the value.

Number of sneakers. Sneakers are permitted left-turn vehicles that exit the intersection at the end of the green phase, usually during the intergreen interval. Equation (4.7), which is based on the results from NETSIM, reveals that the number of sneakers ranges from 1.3 to 2.4 vehicles per cycle. The HCM allows two left turns per cycle. The ARRB indicates that at least 1.5 vehicles per cycle can turn left after the end of the green phase. The State Department of Highway and Public Transportation in Texas (1974) uses a value of 1.6 left turns per cycle as a minimum in capacity analyses. Furthermore, Lin et al. (1984) used the TEXAS Model to develop guildlines for use left-turn lanes and signal phases. They

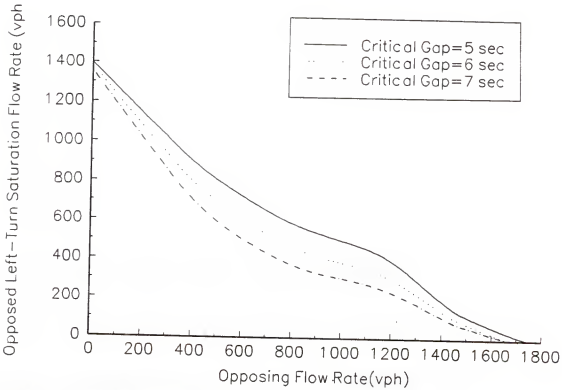


Figure 5.2. Simulated opposed saturation flow rate with different critical gaps.

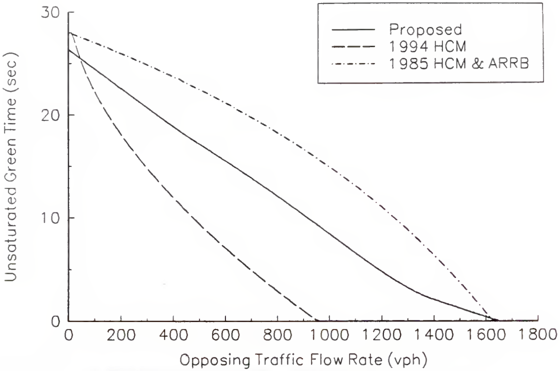


Figure 5.3. Comparison of simulated unsaturated green time with existing models.

found that the number of sneakers is increased up to one left turn per cycle as the opposing traffic flow increases.

Left-turn Factor for Compound Left-turn Protection

The left-turn adjustment factor is based on the type of phasing (protected, permitted, or protected-plus-permitted) and whether left turns are made from exclusive or shared lanes. The left-turn adjustment factor is 1.0 if the lane group does not include any left turns. Basically, left-turn factors account for the fact that these movements can not be made at the same saturation flow rates as thru movements. They consume more of the available green time and, consequently, more of the lane group's available capacity.

The left-turn factors reflect six different conditions under which left turns may be made, as follows:

Case 1: Exclusive left-turn lane with protected phasing
Case 2: Exclusive left-turn lane with permitted phasing
Case 3: Exclusive left-turn lane with protected-plus-permitted phasing
Case 4: Shared left-turn lane with protected phasing
Case 5: Shared left-turn lane with permitted phasing
Case 6: Shared left-turn lane with protected-plus-permitted phasing

Cases 2 and 5 were discussed in previous sections. This section concentrates on the analysis for the other cases.

Exclusive left-turn lane with protected phasing (Case 1). Calculation of the left-turn adjustment factor for a exclusive protected left-turn lane can be treated as a special case of exclusive permitted left-turn lane by ignoring the effects of opposing flow. The opposing flow rate can be set to zero since the left turns are protected. The value of the left-turn adjustment factor can be obtained from Equation (5.1) and (5.2) for single-lane approaches and multilane approaches, respectively, and assuming 1,900 vph of the ideal saturation flow rate for a thru movement.

$s_u = 1{,}615$, $f_{LT} = 1{,}615/1{,}900 = 0.85$, for single-lane approach.

$s_u = 1{,}535$, $f_{LT} = 1{,}535/1{,}900 = 0.81$, for multilane approaches.

Exclusive left-turn lane with protected-plus-permitted phasing (Case 3). The method for analyzing this kind of left-turn operation is similar to the 1994 HCM method. In general, protected-plus-permitted phases for exclusive lanes are analyzed by separating the portions of the phase into two-lane groups. Each portion of the phase is then handled as it would be normally if the other were not present. The protected portion of the phase is treated as a protected phase, and a left-turn adjustment factor appropriate to a protected phase is selected. The permitted portion of the phase is treated as a permitted phase, and the procedure for estimating the left-turn adjustment factor for an exclusive permitted left-turn lane is used. In the calculation of the f_{LT} permitted portion of such phase, the difficulty is in selecting values of g, g_f, g_q, and g_u for using these models. The equation for g_q is indexed to the beginning of the effective green for the opposing flow, and the g_f is indexed to the beginning of the effective green in the subject direction. When leading phasing exists, these equations must be modified to account for shifts in the initiation of green time. Lost time should not be applied to the g_f and g_q since it does not apply to effective green under such a condition, which was discussed previously.

Shared left-turn lane with protected phasing (Case 4). To estimate the left-turn adjustment factor for a protected shared left-turn lane, two critical conditions should be identified. First, the left-turn adjustment factor is equal to 1.0 when the proportion of left turns is zero. Second, the left-turn adjustment factor is equal to that of a protected exclusive left-turn lane (Case 1) if the proportion of left turns is one. The left-turn adjustment factor should range between these two critical conditions. Thus, the left-turn adjustment factor can be approximated by a linear function of the proportion of left turns as follows:

$$f_{LT} = \begin{cases} 1.0/(1.0+0.15P_{L}) & \text{single-lane approach} \\ 1.0/(1.0+0.19P_{LT}) & \text{multilane approach} \end{cases} \tag{5.10}$$

Shared left-turn lane with protected-plus-permitted phasing (Case 6). Similar to Case 3, protected-plus-permitted phases for shared lanes are analyzed by separating the portions of the phase into two-lane groups. Each portion of the phase is then handled as it would be normally if the other were not present. The protected portion of the phase is treated as a protected phase, and a left-turn adjustment factor appropriate to a protected phase is selected. The permitted portion of the phase is treated as a permitted phase, and the procedure for calculating the left-turn adjustment factor for a shared permitted left-turn lane is used. Again, the equations for g_f and g_q must be modified to account for the elimination of lost time.

Incorporation of the Proposed Model into the HCM Procedure

When permitted left turns exist, either from shared lanes or from exclusive lanes, the procedure for analyzing the saturation flow rate is complicated. The basic cases with a permitted left-turn phase are discussed as follows.

Basic Model for Multilane Approaches Opposed by Multilane Approach

Based on the enhancement presented previously, the left-turn adjustment factor for the lane from which permitted left turns are made can be stated as

$$f_m = (\frac{g_f}{g}) + (\frac{g_u*}{g})[\frac{1}{1+P_L(E_{L1}-1)}] + \frac{2n_s P_L}{g} \tag{5.11}$$

Note that the last term in this formulation includes the effect of sneakers, that is, vehicles completing left turns during the clearance or change interval. Meanwhile, it is not necessary to impose a minimum number of sneakers on f_m. The minimum number of sneakers already is embodied in this term.

For multilane groups, left turns impact not only on the shared lane but also on the entire lane group. To take this into consideration, the following relationship should be used:

$$f_{LT} = \frac{[f_m + 0.91(N-1)]}{N} \tag{5.12}$$

where f_{LT} = left-turn adjustment factor applied to a total lane group from which left turns are made, and

f_m = left-turn adjustment factor applied only to the lane from which left turns are made.

When a single (or double) exclusive permitted left-turn lane is involved, $f_{LT} = f_m$. To implement the model of Equation (5.11), it is necessary to estimate the subportions of the effective green phase, g_f, g_q, and g_u. Regression relationships described in Chapter Four are as follows:

1. Compute g_f:

for shared permitted left-turn lanes:

$$g_f = G' \times \exp(-0.3782 LTC^{0.8086} P_{LT}^{0.1943}) - t_L \qquad (5.13)$$

$$0 \le g_f \le g$$

for exclusive permitted left-turn lanes:

$$g_f = 0.0$$

2. Compute g_q:

$$g_q = g - g_u \qquad (5.14)$$

where g_u uses Equation (5.3)

3. Compute g_u*:

$$g_u{*} = \begin{cases} g - g_q & \text{when } g_q \ge g_f \\ g - g_f & \text{when } g_q < g_f \end{cases} \qquad (5.15)$$

4. Compute P_L (proportion of left turns in a shared lane):

$$P_L = P_{LT}\left[1 + \frac{(N-1)g}{(f_s g_u + 4.5)}\right] \qquad (5.16)$$

where $f_s = (875 - 0.625 v_o)/1000 \quad f_s \ge 0$

5. Compute E_{L1}:

$$E_{L1} = \frac{s_o}{s_u} \tag{5.17}$$

where s_u uses Equation (5.2)

6. Compute n_s: using Equation (5.5)

Basic Model for Single-Lane Approaches Opposed by a Single-Lane Approach

The case of a single-lane approach opposed by another single-lane approach has a number of unique features that must be reflected in the model. The most critical of these is the effect of opposing left turns. An opposing left-turn vehicle in effect creates a gap in the opposing flow through which a subject left turn may be made. This can occur during the clearance interval of the opposing queue as well as during the unsaturated portion of the green phase.

Since vehicles do not have the flexibility to choose lanes on a single-lane approach, regression relationships for predicting g_o and g_q also are different from those for the multilane case. Further, for a single-lane approach, f_{LT} is equal to f_m, and P_L is equal to P_{LT}. The basic model for opposing single-lane approaches is, therefore,

$$f_{LT} = (\frac{g_f}{g})+(\frac{g_{diff}}{g})[\frac{1}{1+P_L(E_{L2}-1)}]+(\frac{g_u*}{g})[\frac{1}{1+P_L(E_{L1}-1)}]+\frac{2n_sP_L}{g} \tag{5.18}$$

To implement this model, it is again necessary to estimate the subportions of the effective green phase, g_f, g_q, and g_u, as follows:

1. Compute g_f:

for free queue = 0

$$g_f = G' \times \exp(-0.4544 LTC^{0.9946} P_L^{-0.2383}) - t_L \tag{5.19}$$

for free queue = 1

$$g_f = G' \times \exp(0.1488 LTC^{1.3027}) - t_L \tag{5.20}$$

for free queue = 2

$$g_f = G' \times \exp(0.0964 LTC^{1.3045}) - t_L \tag{5.21}$$

2. Compute g_q: using Equation (5.14).
3. Compute g_u*: using Equation (5.15).
4. Compute E_{L1}:

$$E_{L1} = \frac{S_o}{S_u} \tag{5.22}$$

where S_u uses Equation (5.1).

5. Compute E_{L2}:

$$E_{L2} = \frac{(1 - P_{THo}^{n})}{P_{LTo}} \tag{5.23}$$

Special Cases for Permitted Left Turns

Two special cases for fully permitted left turns must be addressed: a single-lane approach opposed by a multilane approach, and vice versa.

When the subject lane in these cases is the single-lane approach, it is opposed by a multilane opposing flow. Even if the opposing approach is a single thru lane and an exclusive left-turn lane, opposing left turns will not open gaps in the opposing flow. Thus, the special structure of the single-lane model does not apply. The multilane model is applied, except that

$f_{LT} = f_m$. The value of g_f should be computed using the single-lane equations, Equations (5.19-21).

When the multilane approach is considered, the reverse is true. The opposing flow is in a single lane, and opposing left turns conceivably could open gaps for subject left turners. The single-lane model may be applied, with several revisions:

1. g_f should be computed using the multilane equation, Equation (5.13).

2. P_L must be estimated and substituted for P_{LT} in the single-lane model. P_L may be estimated from P_{LT} using the multilane equation, Equation (5.16).

3. f_{LT} does not equal f_m. The conversion must be made using the multilane equation, Equation (5.12).

CHAPTER SIX

MODEL EVALUATION

The main objective of this section is to examine the sensitivity of the proposed models to various traffic conditions. The proposed model, the 1985 HCM, and the 1994 HCM were compared with the simulation results. NETSIM again was used as the evaluation tool. The results of the comparison are discussed in this chapter.

Description of Scenarios

Six different scenarios were considered to assure a comprehensive evaluation procedure. This study essentially concentrates on the left-turn saturation flow rate without the interference of any external factors. The lane geometries are the same in all four approaches for a particular scenario. The right-turn movement is neglected in all six scenarios. The start-up lost time for each phase is assumed to be three seconds, and a change interval of three seconds is employed. The green time is kept constant at 12 seconds for the protected left-turn movements and 27 seconds for the permitted left-turn movements and thru movements. The cycle lengths are kept constant at 60 and 90 seconds for permitted only and compound left-turn protection phasing, respectively. The six scenarios fell into six categories based on intersection configuration, left-turn treatment, left-turn lane use, and the number of thru lanes. A summary of the characteristics of the data is presented in Table 6.1. For the scenarios with exclusive left-turn lanes, the left-turn volume for subject approaches (northbound and eastbound) is kept constant at 850 vph, which is over capacity, so that the saturation flow rate can be obtained. For the scenarios with shared left-turn lanes, the total approach volume in the subject directions is kept constant at 850 vph and 15 percent of that volume are assigned as left turns. The intersection layout and phasing pattern for each scenario are described below in detail.

Scenario 1. The thru movement has a single lane and the left-turn movement has an exclusive lane (Figure 6.1). The signal has a simple two-phase sequence, with phase times as shown in the figure. The left-turn volume is kept constant at 850 vph. The opposing thru-movement volume is varied through a range from 45 vph to 900 vph in increments of 45 vph.

Table 6.1. Summary of the characteristics of the scenarios.

Scenario	Case name	Type of left-turn lane	Left-turn treatment	# of Lanes	Split
1	EM1	exclusive	permitted	1	EW:30 sec; NS:30 sec
2	EM2	exclusive	permitted	2	EW:30 sec; NS:30 sec
3	EC2	exclusive	protected/ permitted	2	EWL:15 sec; EW:30 sec; NSL:15 sec; NS:30 sec
4	SM1	shared	permitted	1	EW:30 sec; NS:30sec
5	SM2	shared	permitted	2	EW:30sec; NS:30sec
6	SC1	shared	protected/ permitted	1	E:15 sec; EW:30 sec; N:15 sec; NS:30 sec

Scenario 2. The left-turn movement has an exclusive lane, and the thru movements occur in dual lanes (Figure 6.2). The signal has a simple two-phase sequence, with phase times as shown in the figure. The left-turn volume is kept constant at 850 vph. The opposing

thru-movement volume is varied through a range from 90 vph to 1800 vph in increments of 90 vph.

Scenario 3. All conditions are the same as in Scenario 2 except that the phasing is changed from permitted only to protected-plus-permitted phasing. The protected left-turn phase is the first phase of the cycle. The concurrent thru and the yielding left-turn movements follow the protected phase (Figure 6.3). The left-turn volume is kept constant at 850 vph. The opposing thru-movement volume is varied through a range from 90 to 1800 vph in increments of 90 vph.

Scenario 4. The intersection is a four-leg intersection with one lane in each direction. No left-turn lanes are present on any approach (Figure 6.4). The signal has a simple two-phase sequence, with phase times as shown in the figure. The total approach volume in the subject directions (eastbound and northbound) is kept at 850 vph, and 15 percent of the volume is assigned as left turn. The opposing volume is varied through a range from 45 vph to 900 vph in increments of 45 vph, and 10 percent of the volume is assigned as left turns.

Scenario 5. The intersection is a four-leg intersection with two lanes in each direction. No left-turn lanes are present on any approach (Figure 6.5). The total approach volume in the subject directions (eastbound and northbound) is kept at 1700 vph, and 15 percent of the volume is assigned as left turns. The opposing volume is varied through a range from 90 vph to 1800 vph in increments of 90 vph.

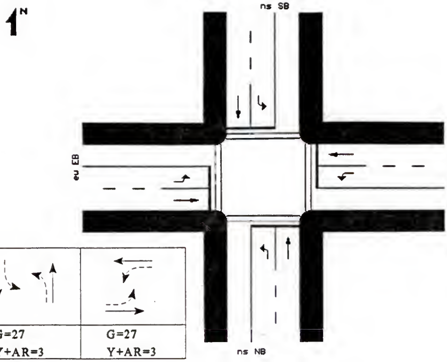


Figure 6.1. Intersection layout and phasing pattern for Scenario 1.

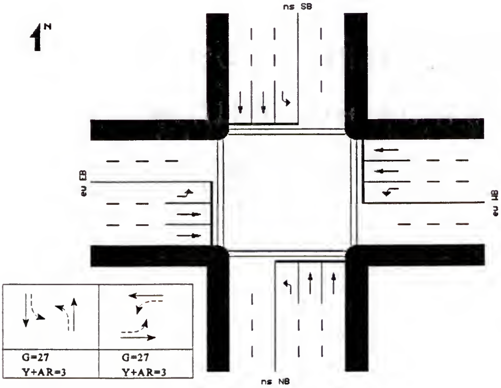


Figure 6.2. Intersection layout and phasing pattern for Scenario 2.

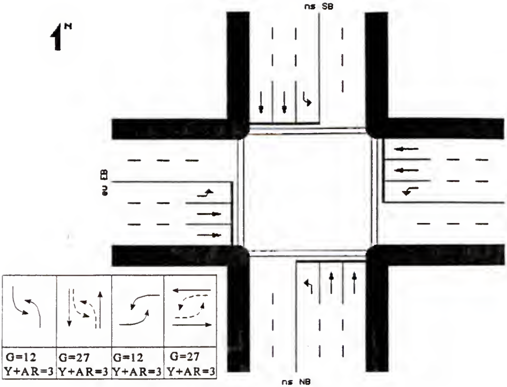


Figure 6.3. Intersection layout and phasing pattern for Scenario 3.

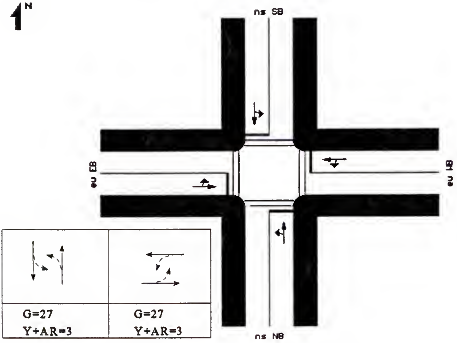


Figure 6.4. Intersection layout and phasing pattern for Scenario 4.

Scenario 6. All conditions are the same as in Scenario 4 except that the phasing is changed from permitted only to protected-plus-permitted phasing (Figure 6.6). The protected phase in the subjective direction is the first phase of the cycle. The concurrent thru and the yielding left-turn movements follow the protected phase. The total approach volume in the subject directions (eastbound and northbound) is kept at 1700 vph, and 15 percent of the volume is assigned as left turns. The opposing volume is varied through a range from 90 vph to 1800 vph in increments of 90 vph.

NETSIM, Highway Capacity Software (HCS) Release 2.3 (implementation of the 1985 HCM methodology), HCS Release 2.4 (1994 HCM methodology), and a special version of HCS which implements the proposed model were run to model the six scenarios described. A database of capacity for the left-turn lane groups of the various models was created and the results were plotted.

Discussion of Results

The results of the computer runs of all the models were plotted with capacity against the varying volumes for all six scenarios. Regression analyses were performed for each scenario and also for the combination of all scenarios. The comparison of models for various scenarios are presented in the following seven figures.

The R^2 value in each scenarios for the proposed model is very high. Most of them are close to 0.98. The overall R^2 is 0.96 of the studied scenarios. These results indicate that the capacities predicted by the proposed model are very close to those estimated by NETSIM.

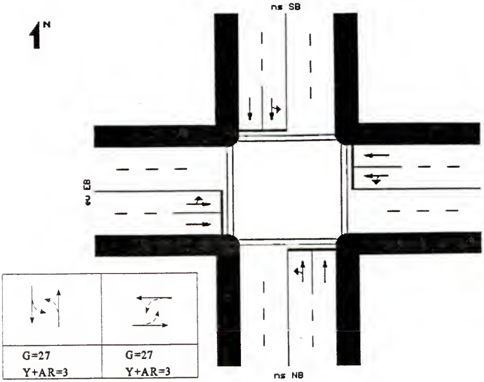


Figure 6.5. Intersection layout and phasing pattern for Scenario 5.

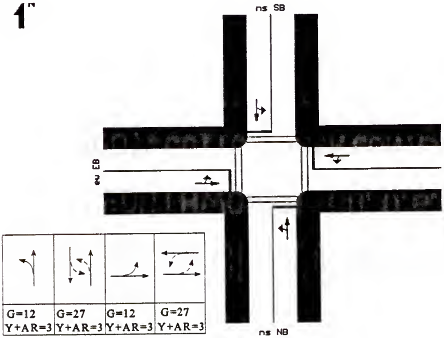


Figure 6.6. Intersection layout and phasing pattern for Scenario 6.

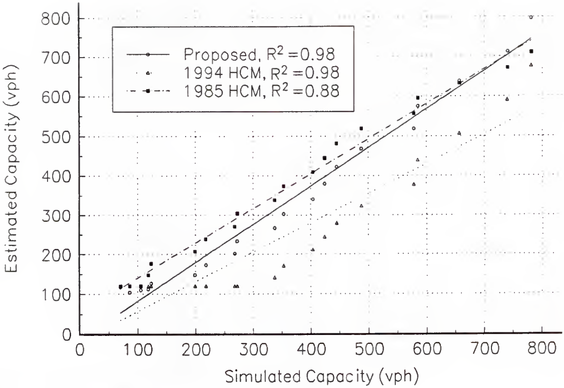


Figure 6.7. Prediction of capacity for Scenario 1.

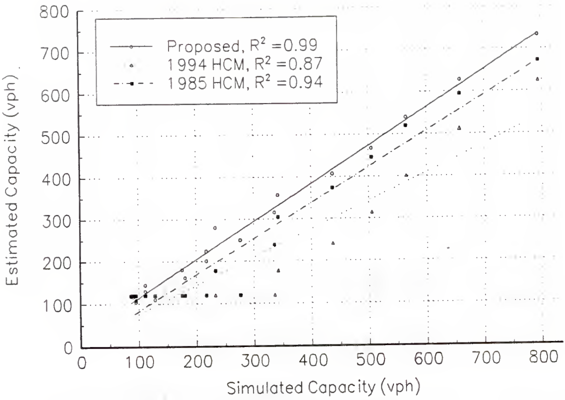


Figure 6.8. Prediction of capacity for Scenario 2.

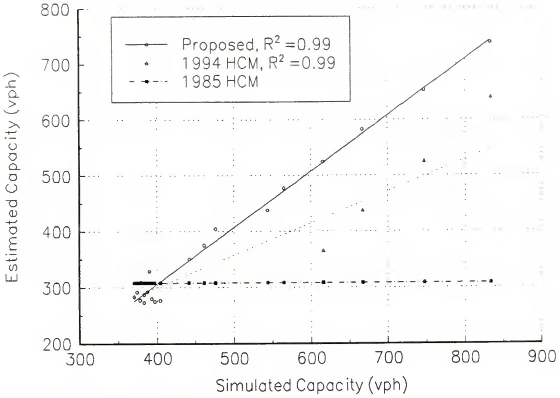


Figure 6.9. Prediction of capacity for Scenario 3.

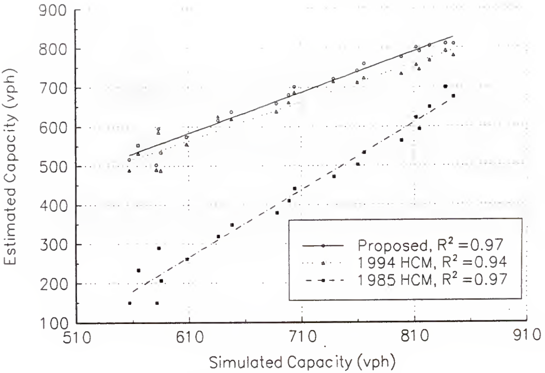


Figure 6.10. Prediction of capacity for Scenario 4.

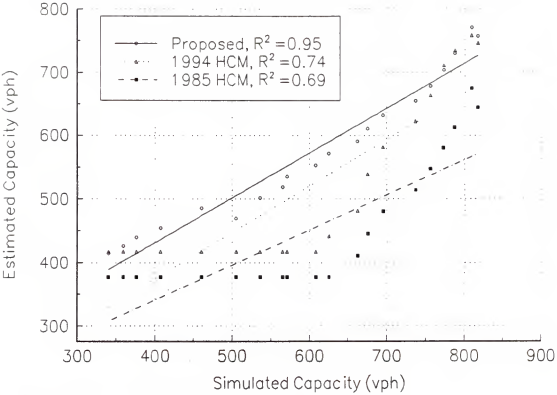


Figure 6.11. Prediction of capacity for Scenario 5.

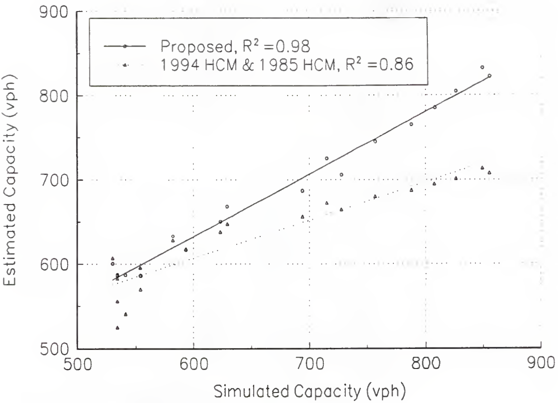


Figure 6.12. Prediction of capacity for Scenario 6.

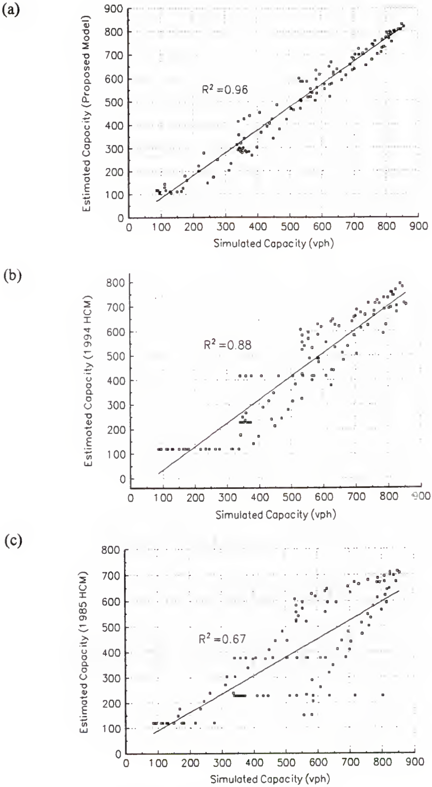


Figure 6.13. Prediction of capacities.
(a) proposed model estimation
(b) 1994 HCM estimation
(c) 1985 HCM estimation

CHAPTER SEVEN

CONCLUSIONS AND RECOMMENDATIONS

This dissertation presents a methodology for exploring considerably more detailed traffic data in NETSIM. By using this methodology, the delay estimates from NETSIM and the HCM were differentiated, and a proposed model for estimating the saturation flow rate of permitted left turns was developed. This model can accommodate both exclusive and shared left-turn lanes in addition to compound left-turn protection. The proposed model was incorporated into draft HCM procedures. A special version of the Highway Capacity Software (HCS) also was developed to allow users to implement the proposed model. The results of the proposed model implementation were compared to the results obtained from the simulation. The proposed model was shown to be a viable alternative for analyzing left-turn flow. The conclusions and recommendations listed below are offered as a result of this study.

Conclusions

The proposed model is intuitively appealing, because it possesses the mathematical structure of existing models and is capable of realistically incorporating all associated factors. The conclusions are listed below.

1. Average vehicle delay is the key indication of intersection performance. It was observed that the definitions of delay are different between NETSIM and the HCM. Furthermore, it was found that, NETSIM estimates values of total delay lower than the HCM.

2. Under high volume conditions, NETSIM predicts incremental delays of about 30 percent less than estimates from HCM procedure. This is because NETSIM specifies less randomness of free-flow speed and arrivals.

3. In modeling the opposed left-turn saturation flow rate, it was found that NETSIM specified a default value of 38 percent for the probability of left-turn jumpers but the HCM assumes no left-turn jumpers. Furthermore, the function to change this parameter does not work in the current version of NETSIM (version 5).

4. The HCM considers the total number of signal phases as an important parameter for estimating the opposed left-turn saturation flow rate. However, it is found in this study that the total number of signal phases is not significantly important to the opposed left-turn saturation flow rate.

5. It was found that the number of left-turn sneakers is a function of the opposing flow rate and ranges from about 1.3 to 2.4 vehicles per cycle. The HCM defaults to a maximum of two sneakers per cycle. By using the proposed model to estimate the number of sneakers, a more sensitive left-turn saturation flow rate can be obtained.

6. The distribution of opposing headways at the stopline after the opposing queue has been discharged plays a key role in left-turn capacity. It was found that the opposing headway at the stopline ranges from 1.06 seconds to 2 seconds and is a function of both the

opposing green time and the opposing flow rate. Consequently, the constant value of 2 seconds assumed in the existing analytical models is overestimated.

7. The time before the first left-turn arrives in a shared lane (g_f) is an important factor in modeling the saturation flow rate of shared permitted left-turn lanes. It was concluded from this study that the parameters affecting the value of g_f include effective green time, left-turn volume, proportion of left-turns, number of opposing lanes, and free-queue maneuver.

8. The methodology developed in this dissertation may be applied to the analysis of other complex stochastic traffic operations. Specifically, it can be used to study traffic operations with gap-acceptance behavior, such as right turns on red and the capacity of stop sign (or yield sign) controlled intersections.

Recommendations

Three sets of recommendations emerged from this research. First, there are recommendations concerning the application of the proposed models; second, there are suggestions for improvements to the NETSIM model; third, there are indicated areas that appear to warrant future research studies.

Model Application

Three recommendations for applying the proposed model are listed below.

1. Based on the limited situations investigated in this study, the proposed model produces reasonable performance estimates for left-turn lanes. Logically, the next step would be an empirical investigation to validate this methodology and identify any needed modifications.

2. The HCM is currently the most widely used procedure for analyzing intersection operations. A recommendation is that the proposed model developed herein be an alternative method for analyzing opposed left-turn flow for use in situations where maximum compatibility with NETSIM is required.

3. The proposed model can be used in signal-timing optimization software. When timing is to be optimized for a given performance index, the capacities of every approach must be determined. The proposed model could provide a good estimate of left-turn lane capacity that can be used to determine signal timing. Three examples of such software are the Signal Operations Analysis Package (SOAP), TRANSYT-7F, and PASSER II.

NETSIM Improvement

In its current status, the NETSIM version 5 is fully operational, but there are some limitations in its application. Three functional enhancements are suggested below.

Average headway. NETSIM embodies a default value of 2.2 seconds for the mean headway between vehicles discharging from a standing queue. Users also are capable of specifying this parameter, but it would be better to have 1.9 seconds of headway as the default value, since a 1,900 pcphgpl of ideal saturation flow rate has been reported in recent research (Roess, Ulerio and, Papayannoulis, 1990) and is currently used by the 1994 HCM.

Two-way left-turn lanes. Two-way left-turn lanes are those where left turns are allowed from opposite directions using the same left-turn lanes. NETSIM does not simulate this kind of left-turn lane.

Intersection logic. In earlier versions of NETSIM, left turners waiting for a gap in the opposing traffic would stop and wait at the stopline of a node. In the new version (version

5) with better intersection logic, left-turn vehicles may proceed into the intersection and stop in the middle of the intersection until a gap is available. This feature better represents traffic behavior within an intersection. However, this behavior is not desirable for all left turners in an intersection. In the simulation only a portion of the left turners should be able to perform this maneuver.

Recommended Future Research

Two specific topics for further research are listed below.

1. An assumption is made in the development of the proposed model. The arrival type is assumed to correspond with totally random arrivals (i.e., an isolated intersection). An investigation into the effect of the progression quality on the left-turn capacity will be beneficial when dealing with real-world situations.

2. The proposed model developed in this dissertation is based on pretimed signal control operations. Actuated signal control is becoming more widely used. Further research that aims at recognizing the influences of actuated signal control on left-turn saturation flow rates is recommended.

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BIOGRAPHICAL SKETCH

Jer-wei James Wu was born in the City of Taichung, Taiwan, Republic of China, on August 23, 1965. In June, 1987 he received his Bachelor of Science degree from the Department of Civil Engineering, Chung-Yung University.

Subsequently, he was called to armed service in the Republic of China Army. Upon release from the Army in 1989, he accepted a position as a construction engineer at the Chun-Yung Steel Construction Company. A large part of his work involved steel bridge construction and Taipei Rapid Rail construction.

In 1990 he enrolled at Florida International University in Miami where he studied civil engineering, specializing in transportation. During this time, he was a research assistant with the Department of Civil Engineering working on a Federal Transit Authority project. He was awarded the Master of Science degree in June, 1992.

Mr. Wu moved to Gainesville, Florida, in August, 1992. He entered the doctoral program in civil engineering at the University of Florida. During the course of his studies, he did research on technical support and software maintenance for Highway Capacity Software.

He won the Annual Florida Section Institute of Transportation Engineers Student Paper Award in 1994. The next year, he was awarded the University of Florida International Student Academic Award.

Mr. Wu is married to Lei and has one son, Derek. Mr. Wu is a member of the Institute of Transportation Engineers, the American Society of Civil Engineers, and the China Road Federation.

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Engineer of Civil Engineering

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